OCCASION

This publication has been made available to the public on the occasion of the 50th anniversary of the United Nations Industrial Development Organisation.

DISCLAIMER

This document has been produced without formal United Nations editing. The designations employed and the presentation of the material in this document do not imply the expression of any opinion whatsoever on the part of the Secretariat of the United Nations Industrial Development Organization (UNIDO) concerning the legal status of any country, territory, city or area or of its authorities, or concerning the delimitation of its frontiers or boundaries, or its economic system or degree of development. Designations such as "developed", "industrialized" and "developing" are intended for statistical convenience and do not necessarily express a judgment about the stage reached by a particular country or area in the development process. Mention of firm names or commercial products does not constitute an endorsement by UNIDO.

FAIR USE POLICY

Any part of this publication may be quoted and referenced for educational and research purposes without additional permission from UNIDO. However, those who make use of quoting and referencing this publication are requested to follow the Fair Use Policy of giving due credit to UNIDO.

CONTACT

Please contact publications@unido.org for further information concerning UNIDO publications.

For more information about UNIDO, please visit us at www.unido.org

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
Vienna International Centre, P.O. Box 300, 1400 Vienna, Austria
Tel: (+43-1) 26026-0 · www.unido.org · unido@unido.org

RESTRICTED

16754

DP/ID/SER.A/992
28 March 1988
ENGLISH

LOW-COST BUILDING MATERIALS TECHNOLOGIES AND CONSTRUCTION SYSTEMS

DP/RAS/82/012

Technical report: Typhoon Assessment Manual*

Prepared by the United Nations Industrial Development Organization acting as executing agency for the United Nations Development Programme

Based on the work of Mr. Geoffrey N. Boughton
Technical Consultant on Typhoon Resistant Construction

Backstopping Officer: B. Der Petrossian, Chemical Industries Branch

United Nations Industrial Development Organization
Vienna

*This document has been reproduced without formal editing.

V.88-23595

TYPHOON DAMAGE ASSESSMENT MANUAL

TABLE OF CONTENTS

G N BOUGHTON

Senior Lecturer in Structural Engineering
Curtin University of Technology

This document is a reference for a Seminar Workshop on Assessment of Typhoon Damage, organised by the UNDP/UNIDO Regional Network in Asia for Low Cost Building Materials, Technologies and Construction Systems.

I presents analysis methods for the determination of wind speeds from structural damage and the determination of wind loads from wind speed data. It also presents practical guidelines on the implementation of damage assessment programmes and on the production of reports that will provide valuable information for upgrading the structural performance of buildings in order to resist future typhoons.

1. INTRODUCTION

Tropical cyclones, typhoons and hurricanes are large scale meteorological phenomena that affect many tropical countries. The strong winds, torrential rain and storm surge effects that accompany the passage of a typhoon have the potential to cause significant damage to mankind's housing, activities and livelihood. In many cases people's lives may come under threat due to the structurally inadequate nature of the shelter provided.

In many parts of the world, the rainfall provided by typhoons is vital to the agriculture of neighbouring regions. Thus, if the life and livelihood threatening potential of typhoons can be significantly reduced, the passage of a typoon could be regarded as a benefit rather than a costly chance of nature. The siting of shelter above storm surge and flood zones will minimise the threat posed by both torrential rain and storm surge effects, and the adequate structural design of shelter will ensure that it is capable of withstanding the effects of the strong winds.

A dilema facing many tropical countries is that the structural action of even simple houses under typhoon loads is extremely complex. Normal structural engineering assumptions include neglecting the structural strength of cladding in consideration of the stability of the building, and the provision of a structural skeleton which carries loads by bending and axial forces. The use of those assumptions in the design of housing can greatly increase the cost of shelter, so alternative design methods must be employed.

In the past, design by experience was an excellent guide. If a system managed to withstand a number of severe events, then it was judged a good one, whereas those that were significantly effected were discarded. The process proved successful, but in regions where significant events can occur at fifty year intervals, the evolution of good designs may take many generations. With the diversity of building materials available today and the speed with which new building products are developed, the natural evolution of good designs must be accelerated.

Three techniques are at the disposal of people charged with the production of safe housing to ensure that house designs will be structurally sound.

(i) Detailed structural design of complete houses for typhoon wind loads.

(ii) Exhaustive structural testing of building components or even complete buildings.

(iii) Scientific evaluation of the damage caused to housing by typhoons and rapid implementation of the lessons learned into future house designs.

The first alternative requires a large resource of skilled technicians that can check the structural design of housing and ensure adequate performance, yet at a reasonable cost. In many typhoon-prone countries, the building industry does not have the financial capability to carry that resource.

The second alternative also requires a substantial structural engineering resource. Test data is necessary for the adequate design of housing, but as the Australian experience in Darwin proved, great care needs to be taken in the correct simulation of the nature of typhoon loads. Cyclone 'Tracy' in December, 1974 caused damage to a number of building systems that had been tested with statically applied loads. Many of the failures were associated with fatigue damage and occurred at loads significantly less than the static test load. The requirement to perform tests for typhoon resistance under cyclic loading leads to an increase in the cost of the tests and time taken for them.

The third alternative complements the other two in that it provides valuable feedback on the performance of building systems, but it also has potential to be used as the principal tool in the formulation of typhoon-resistant housing. In effect, every typhoon load tests many thousands of houses, and under undisputable field conditions. If the data on the perfoımance of the houses tested can be assembled and evaluated, it has the potential to save much laboratory testing, and analysis work in the design of housing.

This monograph addresses the philosophy and techniques necessary for the effective assessment of damage after the passage of typhoons, and the subsequent steps for incorporation of good construction practice in future house designs.

1.1 Aims Of Damage Assessment Programmes

The principal aim of any damage assessment programme is the systematic documentation of the performance of housing.

Unless the programme is commissioned by an individual or group with highly specific interests, the report should be designed to contain the following items which will make it useful to a wide readership.

- documentation of meteorological data on the typhoon including central pressure at the time it crossed the coast, estimates of maximum wind speeds at locations where damage is reported and estimates of the duration of the destructive winds.
- documentation of structural failures in buildings which should include estimates of loads in damaged components and identification of critical elements in those buildings.
- documentation of structural systems which have performed satisfactorily under the typhoon winds.
- recommendations on the implementation of changes to building designs or construction system.
- recommendations of work for further research activities.

The importance of including all those items cannot be over emphasised. Each of those items enables comparison of building performance between countries and regions. The recommendations therefore have a much wider applicability than just the affected locality

A number of subsidiary aims relate to the philosophy adopted for the assessment at the planning stage.

Until a comprehensive data base of information on the performance of many different construction systems has been established, the assessment programme should aim at evaluating as many different types of construction systems as possible. The effort required to achieve this aim will necessarily mean that less detail will be available for any one construction system.

After the establishment of a comprehensive data base, later assessment programmes can aim to extract very detailed information on just those systems that appear to be less able to resist typhoon winds.

In the planning of an assessment activity, the organisers should aim to involve local builders and/or building inspectors. This will enable particular problems to be identified as either common local practice, common practice throughout the region, isolated cases of poor workmanship or isolated cases of poor design practice.

1.2 Planning Activities For Damage Assessment

Most regions have contingency plans for aid to victims of natural disasters. These are generally made well in advance and can be implemented very quickly following the passage of a typhoon. As it is important that damaged buildings be assessed prior to clean-up and re-construction activities, it is also necessary for assessment activities to be planned well in advance.

For each region a series of contacts should be established as shown in Figure 1.

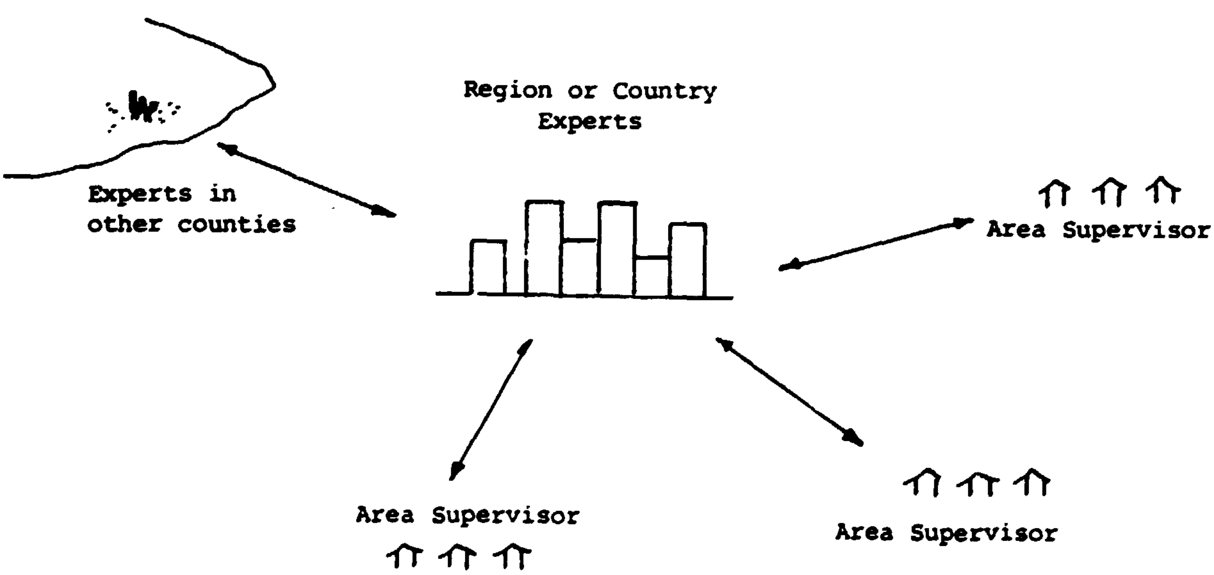


Figure 1: Contacts For Planning Assessment

(a) Area supervisors may be located in major provincial towns and could be government building supervisors or officers of Local Government organisations with a knowledge of building systems. Alternatively, police officers or other emergency services officials may be used.

They should be briefed as to the benefits that could flow to their area in the long term as a result of proper typhoon damage assessment. They should also be given contact details for the region or country assessment experts that will enable them to make early ccntact after the passage of a typhoon.

(b) Region or country experts will generally be centrally located in major cities and may include engineers or architects from universities, building research establishments, or Government departments responsible for the implementation of safe building systems. There may be two or three senior persons and a number of competent supporting technical people that would all be regarded as region or country experts. One of these persons should be chosen in advance to lead the group and be entrusted with the final responsibility of deciding whether an investigation will be performed, and if so, at which level.

(c) Experts in other countries may be able to offer skills in specific types of building systems or types of typhoon damage. They may be called upon by the leader of the regional or country experts to assist in the assessment.

1.2.1 Contingency Plans

These are plans that can be drawn up well in advance of the typhoon season and will establish the following responsibilities.

(i) Who will contact whom after the damage of an event is apparent.

(ii) Who will decide on the scale of the investigation - a number of alternatives are available and outlined in Section 1.2.2.

(iii) Who will make arrangements for practical details such as

- supply of film and cameras
- organisation of accommodation
- organisation of trasnport
- supply of food and water
- supply and arrangement of transport

(iv) Finance for investigation

It always pays dividends to have negotiated allocation of funds for assessment based on the potential of the programme in general. If it is left until after a typhoon has caused damage, the days or weeks it takes to organise finance and justify requests can severely erode the effectiveness of the programme.

Copies of the contingency plan together with a list of area supervisors and their telephone numbers should be distributed to each country or regional expert at the commencement of each typhoon season.

1.2.2 Scale Of Investigation Mounted

This is a direct consequence of the scale of damage caused by any particular event.

(i) Small Scale Operation:

Where a typhoon has caused damage centred on only one town or area, and the extent of the damage is not very severe, then it may be possible to mount a small scale operation.

Typically two or three technical experts may visit the area for two or three days, and work alongside the area supervisor. Under such circumstances, there is not usually a drastic shortage of food and accommodation and general local available facilities can still operate. This makes the organisation of small scale operations simple and travel requirements can be light.

(ii) Concentrated Medium Scale Operations:

Where a typhoon has caused damage centred on only one town or region, but the extent of the damage is moderately severe, the scale of the assessment operation mounted must be increased.

Because the damage is concentrated in one area, transport and logistics do not become a major problem but because there is much more work to be done than in the small scale operation, generally a longer stay is required. Three or four experts may be required for one to two weeks in some cases. As the damage is severe, local support facilities are generally not available, and so frequently the investigation team must supply its own accommodation, food and support personnel.

(iii) Widespread Medium Scale Operations:

Where a typhoon has caused damage over a wide area, but the scale of the damage is not very severe then a widespread medium scale operation must be mounted.

Where the damage is not very severe, the clean-up operations will generally be achieved quite quickly. There is little alternative but to utilize a large assessment team, widely spread for a few days. Typically up to ten experts operating as five teams may operate in the field for two or three days, then spend one day collating information together in the field. It may be necessary to follow-up with more detailed work in one or two locations. This type of operation often presents problems with transport, particularly if the damaged region is distributed over a number of islands. However, because of the nature of the damage, local facilities can generally be relied upon for food and accommodation.

(iv) Large Scale Operations:

Where a typhoon has caused severe damage to a large community such as a city or over a wide area, a large scale operation must be mounted. These require a very substantial organisational input.

The Australian experience of cyclone 'Tracy' in Darwin, 1984 caused very severe damage to a city which was a political, commercial and administrative centre. The clean-up operations took over one year, and reconstruction, more than three years. The damage assessment programme involved more than fifty experts and was spread out over one year. Some experts were on site for as little as one day, other for as long as one month. Frequently laboratory experiments were used to confirm the findings of the field investigation. The final report ran to three volumes.

Organisation of large scale operations must mobilise a large number of experts, frequently more than ten, and co-ordinate their operations. Often transport arrangements are difficult to implement due to the poor condition of roads and bridges after severe typhoons. In many cases, accommodation cannot be provided locally so must be carried with the assessment team.

1.2.3 Role Of The UNIDO/UNDP Regional Network

While it is recognised that initially the contingency plans for each country will be very different due to the different nature of the transportation difficulties encountered, the network offers a vehicle for sharing assessment experiences and the generation of contingency plans with wide ranging applicability.

Through encouraging a dialogue between the assessment programme leaders in each country, the Network will foster a regional responsibility to the assessment of damage. In this way, reports will be produced that will benefit all the member countries.

The Regional Secretariate can effectively disseminate the data produced by assessments within one member country to all the other member countries, and encourage the rapid and widespread implementation of changes to building systems when recommended by an assessment programme.

1.3 Practical Aspects Of Damage Assessment

As alluded to in the previous section, the damage that has disrupted a whole community and has caused implementation of an assessment programme also hinders the assessment in a number of practical ways.

1.3.1 Weather

With the possible exception of typhoons that occur very late in the season, the weather following the passage of a typhoon can be characterised by high humidity and widespread rain. This has a number of implications for damage assessment work.

Special arrangements must be made to store completed notes, tapes or films in such a manner that they are protected from excessive heat and humidity. Aluminium boxes containing sachets of silica gel are ideal for this purpose.

Temporary accommodation should incorporate a drying area for drying out clothes and equipment that get wet in the course of the assessment programme.

Note-taking facilities should be consistent with work in the rain. A small tape recorder that can be kept in a pocket is ideal for this, as it can be protected in a plastic bag and used even in quite heavy rain.

The weather also has implication for transport and accommodation.

1.3.2 Communication and Transportation

Communication and transport facilities are often disrupted as much as buildings during the course of tropical cyclones. Strong winds can damage radio and microwave communication links and torrential rain can cause flooding which can cut roads, damage bridges and cause erosion or softening of unsealed airfield runways.

This will make communications difficult in the early stages after the occurrence of the typhoon. Where possible use may be made of some emergency communication networks such as those operated by the police or military. Problems in communicating with responsible people in the damaged areas can lead to wildly inaccurate press reports of damage soon after the occurrence of a typhoon. Little weight should be attached to early reports provided by inexperienced observers.

Difficulties can also be experienced in moving to damaged areas and within regions recently affected by typhoons. Light aircraft can be used to good effect, but frequently due to low cloud cover are very restricted in their operation.

Vehicular transport may often be difficult due to the presence of water on roads and flood damage. Access to reliable vehicles may be difficult unless the area supervisor can co-ordinate assessment activities with relief supplies.

Problems with transportation are generally not severe for small scale assessments but can become very limiting with large scale and widespread medium scale operations.

1.3.3 Supplies and Accommodation

Where severe damage has occurred, widespread accommodation shortages result. Compromises have to be made in the standard of accommodation accepted.

Again, good relationships with emergency services can assist in this regard. Where the size of the assessment team is not large, it may be possible to be included in relief accommodation. This may include camping in schools or other public buildings.

Where the assessment operation is large, it may be possible to join with a temporary military camp. (These are usually established close to damaged areas to assist with clean-up operations). Where this is not possible an independent camp must be established. This should include sleeping areas, working and cooking facilities, a drying area, and if possible a well lit area for processing of collected data at night times.

Much time in the assessment can be saved, if systematic documentation procedures can be implemented each day. This will enable decisions to be made with respect to the goals for the next day and the effective assignment of manpower resources.

Where an independent camp has been established, extensive use of batteries to power torches, tape recorders, cameras etc will be made. A good supply of fresh batteries must be maintained. Additionally, these circumstances, much of the food must be brought in with the assessment team. At least two people will be required to keep the camp operational and supplied.

1.3.4 Security

The assessment operation represents a substantial investment of time on the part of the assessment technicians and money for transport and accommodation. The investment produces data on the performance of buildings. The exposed films and filled cassettes which constitute the raw data of the assessment, therefore are very valuable and should be stored in safe places.

At the end of each day, notes should be taken on the work done by each technician. These notes should include details on the type of houses inspected and the cause of the first failure. They will enable planning of future work so that the aims of the assessment programme can be fulfilled. These notes also have value in that they will be a major resource in the production of the final report. They should be treated with the same care as the exposed films and filled cassette tapes.

1.4 Social Aspects of Damage Assessment

People who have just experienced the trauma of a typhoon are under a great deal of stress. They may have lost loved ones, friends or relatives and almost certainly would have lost belongings, crops or livelihood and be under financial pressure. This stress and pressure which pervades the whole community must be taken into account when planning and operating an assessment programme.

Another very unfortunate effect of typhoons on communities is that often locking of damaged houses and stores may occur. This also has serious implications for the assessment programme, in that all trespassers are regarded with suspicion.

1.4.1 Protection of Property

Because of the occurrence of looting on damaged properties, individuals may be extremely protective of their property. In some cases this will extend to the use of savage dogs or the use of weapons to safeguard their belongings. Embarrassment and possible injury to the damage assessment team can be avoided by following a simple rule.

ALWAYS ENTER PRIVATE PROPERTY ACCOMPANIED BY AN OWNER OR OCCUPIER OR A POLICE OFFICER

This will ensure that the motive for the inspection is above suspicion and that there can be no accusations of improper behaviour later. The owner/occupier can often provide valuable information about the construction of the building, the history of the damage, and plans for restoration their presence is an asset to the investigation.

1.4.2 Sensitivity to People's Social Needs

If the owner/occupier is available to escort the investigators through his damaged building, as indicated above, he/she will be able to provide much valuable information to the study. They will also want to talk about many things that are of no direct interest to the study such as damage to furnishings, injuries to people and loss of livestock or pets.

While such discussions undoubtedly slow the progress of the study, social workers indicate that a good listening ear is one of the things that people under stress in those situations need most. It pays dividends to take a small amount of time to listen to their story and to "meet the family" if such an opportunity is provided.

In establishing contact with the owner/occupier, it is necessary to give a brief and simple explanation on the 'purpose of the study and the importance of their building to that work'. For example..

"Good afternoon - do you live here?"
"It seems to be rather badly damaged - I am sorry to see that".
"I am part of a team of people who are examining the damage to houses in this area. Our work will help to make houses stronger for future typhoons. The damage to your house looks particularly interesting. May we have a closer look at it?"
"Thank you - my name is ...... etc".

Such a dialogue establishes a basis for further conversation and concisely indicates the purpose of the visit. Some experience will be necessary to establish the fine balance between being interested in the people, and becoming involved in very complex social problems.

It is a good idea to take notes of any requests that are made for supplies, equipment or medical care and pass them on to emergency services at the end of each day.

1.4.3 Respect for Authority

Often the emergency service operations will be co-ordinated by a senior police or military officer. These people must carry a reasonable responsibility for the activities within the disaster area.

IT IS VERY IMPORTANT THAT THE INVESTIGATING TEAM REPORT TO THE EMERGENCY SERVICES CO-ORDINATOR AS SOON AS IT ENTERS HIS AREA.

This serves the following important functions.

(i) It identifies the investigating team and the aims of the assessment programme. This will ensure that the emergency services will not mistake the team for "looters". It is important that the co-ordinator of emergency services understands the aims and benefits of the assessment programme.

(ii) It establishes a dialogue between the emergency services and the investigating team. This is of mutual benefit. The co-ordinator of emergency services generally has an accurate knowledge of the damage pattern within the area. (He has often covered the complete area himself in a helicopter.) This information will assist in the correct placement of the assessment team. In conducting the assessment, it is possible that people in the community will make known needs that can in turn be passed back to the emergency services.

(iii) Later the relationship established and the progress of the assessment may be used to gain access to emergency service transport to move the team into more remote areas.

It is a wise diplomatic move to forward a copy of the completed report to the co-ordinator of emergency services with whom you have worked in the disaster area. He can then appreciate the value of the assessment and can see how his own involvement assisted.

1.5 Technical Aspects of Damage Assessment

Having overcome the practical aspects of the planning and logistics associated with the execution of a damage assessment programme, and the social problems associated with working in a disaster area, it then remains to perform a systematic assesssment to achieve the initial goals.

The rest of this monograph outlines the skills and techniques necessary to achieve the stated aim of the assessment. While the presentation is in a logical order, it is not a requirement that the assessment be executed in the same logical order.

Often it is more convenient to start at a point and work through an area assessing large buildings and small buildings in the order in which they are encountered. In the same way, as simple structures which may give an indication of the wind speed are encountered they should also be assessed.

Towards the completion of the assessment period, more emphasis can be given to aspects that have not been successfully covered.

For example, if after two or three days a very representative sample of housing has been inspected, but few large buildings and estimates of wind speed are still uncertain, technicians may bypass housing in search of large buildings or simple structures to analyse for wind speed.

2. ASSESSMENT OF WIND LOADS

In Chapter 1 it was noted that a typhoon effectively load tests a large number of structures. In performing tests under laboratory conditions it would be unthinkable not to determine the ultimate load on the test piece.

Unfortunately, there are very rarely calibrated load cells of providing rings installed in the buildings that are inspected during the damage assessment, yet it is vital to evaluate the load at which failure would have occurred in order to make the data relevant to buildings in other locations. Loads therefore have to be inferred from the physical setting of the building and the estimated wind speed and direction at that position.

The following steps are involved in the assessment of wind loads on each building inspected.

(i) Estimation of the regional wind speed which is a property of the typhoon itself, and the location of the building with respect to the eye of the typhoon.

(ii) Modification of the regional windspeed to produce a peak gust velocity for the location based on the roughness of the land upwind of the building site.

(iii) Modification of the peak gust velocity to account for the effect of topography immediately upwind of the building location.

(iv) Conversion of peak gust velocity to a load using appropriate pressure coefficients for the structure and wind direction corresponding to the winds which appear to have caused the damage.

These calculations do not have to be performed on site at the time of inspection, but notes should be taken in sufficient detail to enable calculation to proceed at the time the report is produced. The notes should include the following details.

(1) Direction from which the wind was blowing at the time of first failure.

(2) Shape of the profile of the building normal to the wind direction in (1). The eaves height and position of open windows and doors are of particular interests.

(3) Topography immediately upwind of the structure in the direction given in (1) above. If possible, numerical estimates should be given e.g. approximately halfway up a rise of height 10m, slope 1 in 4.

(4) Terrain description for the area immediately upwind of the structure. In rural settings the presence of hedges, wind breaks of other buildings is of particular importance e.g. fifty metres from rice fields that are very open. Space between rice field and building, covered in sparse low (1m high) bushes. One small shed approximately 15m away.

The processing of this information is performed using the methods indicated in Sections 2.3 and 2.4.

2.1 Regional Wind Speed Assessment

The large size of typhoons is an advantage to the assessment of regional wind speeds. In terrain with uniform properties, the wind speeds at different locations can be related to their positions relative to the eye of the typhoon. Measurements or estimates of wind speed from a number of different locations can be correlated using well established relationships.

2.1.1 Correlation Of Wind Speed Between Locations

The wind speed at a particular location and time during a typhoon can be expressed as a function of the central pressure, radius of maximum winds, the forward speed of the typhoon, location relative to the position of the eye, the terrain surface, the height above the surface, latitude, topography and other meterological factors. The wind speed has contribution from the rotation of the typhoon about its eye and from the forward movement of the whole meteorological system. Empirical relationships have been established by a number of different authors to relate these two velocities, the most recent have been presented by Atkinson and Holliday (1975), Gomer and Vickery (1976), Tryggvason (1979) and Georgiou, Davenport and Vickery (1983). The equations presented below for the ten minute mean wind speeds over the sea at a height of ten metres are typical.

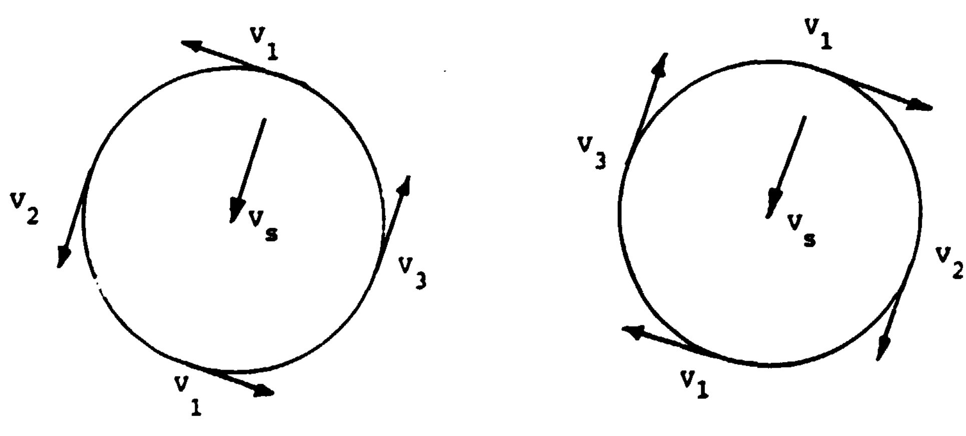
$$V_1 = C\sqrt{p_c - 1010}\left(\frac{R}{r}\right)^k$$

$$V_2 = V_1 + 0.5V_s$$

$$V_3 = V_1 - 0.5V_s$$

where V_1, V_2, V_3 maximum ten minute mean wind speeds (m/s)

p_c	central pressure (mb)
r	radius from centre of eye to point under consideration (km)
R	radius of maximum winds (km)
V_s	forward speed of typhoon (m/s)
C, k	constants obtained by data fitting



Northern Hemisphere   Southern Hemisphere

Figure 2 Wind Speeds In Typhoons

Figure 2 shows that wind speeds are generally higher to the left of the eye in Southern Hemisphere events and to the right of the eye in Northern Hemisphere events.

The calculations using the three relationships given above are complicated by the changes to the wind field that occur as the typhoon makes landfall. Figure 3, taken from a recent report on an Australian tropical cyclone shows that the eye started to decrease in size as soon as it crossed the coast. (Reardon, Walker and Jancauskas, 1986).

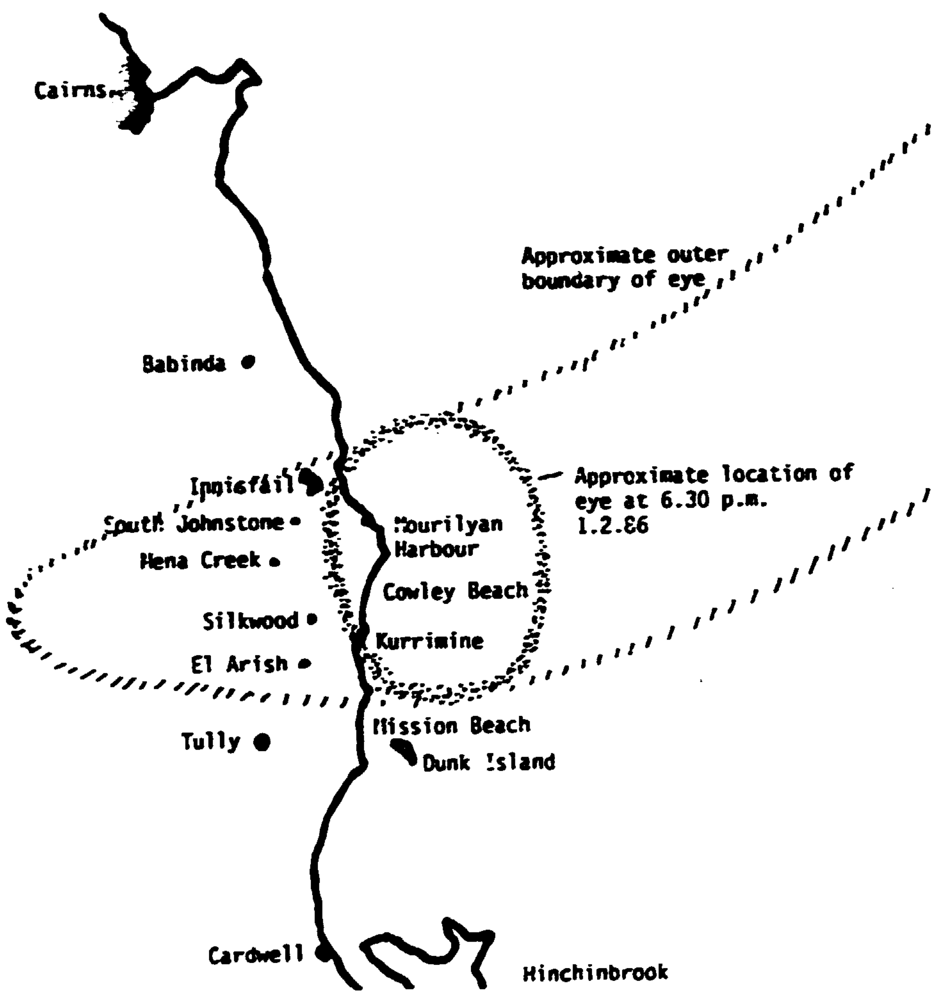


Figure 3 Eye Position As Cyclone Winifred Crossed The North Queensland Coast

The change in size of the eye causes a reduction in R and this is accompanied by an increase in p and often by an increase in V_s. The net result is a reduction in the maximum wind velocities calculated. This is consistent with field observation. It is to be noted that these relationships have been developed for winds over the sea, and should be used with caution when extrapolating velocities for large distances over land.

It is possible to locate the position and size of the eye with a reasonable amount of certainty by recording comments from occupants interviewed during the course of the investigation. Other information can often be obtained by flying over the damaged area, and noting the direction in which most trees had fallen. Where trees have fallen in opposing directions there is a high probability that the centre of the eye passed over that point. Figure 4 illustrates the usefulness of fallen trees in locating the path of the eye.

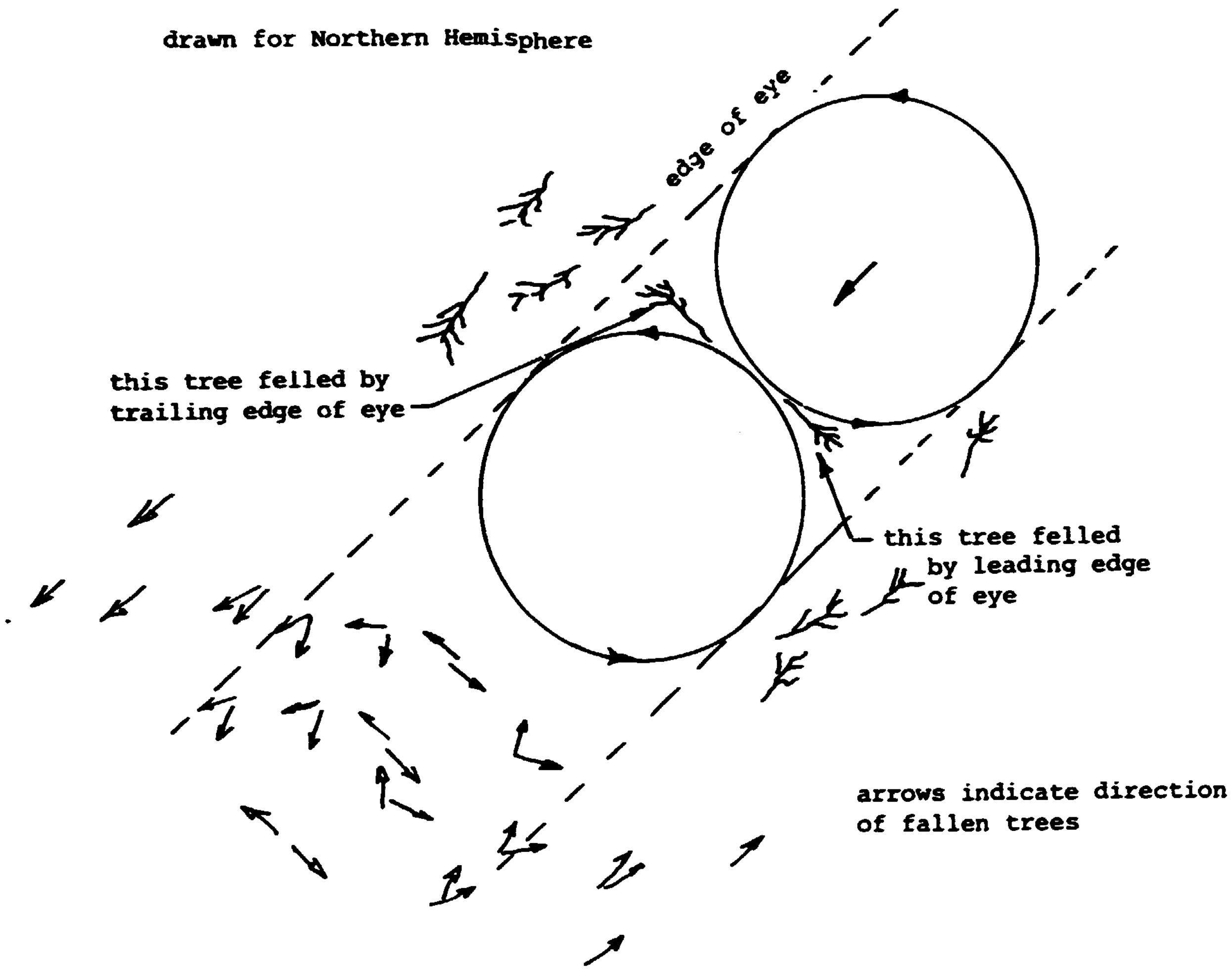


Figure 4 Locating Eye Position From Tree Damage

Establishing the path of the eye at an early stage of the investigation can be advantageous as it can be used to locate communities in which damage is likely to be most severe.

2.1.2 Determination Of Wind Velocities

The techniques described in the preceding subsection can be used to extrapolate velocity measurements or estimates, but they rely on a number of locations at which velocities can be determined. Frequently the determination of these velocities can present a major problem.

In the investigation of typhoon damage three alternatives are sometimes available for the estimation of wind speeds at a given location.

(i) Direct Measurement

Where the typhoon has passed close to a major weather station it is possible that a continuously recording anemometer may have a readable trace. Many types of anemometers have non-linearities at high velocity, so extrapolation of calibration curves should be carried out with extreme caution.

Dines anemometers have proved reliable in recording typhoon strength winds and have a response period of approximately three seconds.

(ii) Inference From Damage To Vegetation

Experienced meteorologists can gauge wind velocity fairly accurately by the extent of damage to certain types of trees. This has led to the development of a number of subjective damage scale such as that proposed by Amadore et al (1985).

(iii) Examination of damage to simple structures. Simple structures which can be easily analysed to find the failure load, can be used to determine upper and lower bounds on wind velocities at the location and height of the structure. Care should be exercised in the choice of these structures and the type of damage, to ensure that it was wind alone that caused the damage, and that the aerodynamics and structural action are are well understood. Some examples of this type of investigation are presented in subsection 2.1.3.

Where possible many independent estimates of regional wind velocities should be obtained. This will enable the use of extensive cross checking of wind speeds and hence wind loads.

2.1.3 Ground Truthing Wind Speeds With Simple Structures

A number of different types of common simple structure can be used to define upper and lower bounds on wind velocities. However, there are a number of pitfalls associated with their analysis.

The analyses presented in this subsection will good estimate of the upper and lower bounds on wind speed at the height of the structures analysed.

The corrections given in sections 2.2 and 2.3 should then be used to relate the wind speed at the structure height to the commonly used regional wind speed equivalent to speed at a height of 10m in open country.

The techniques outlined in this section also serve to introduce the process of analysis. The process will be expanded further in chapters 3 and 4.

(A) Road Signs

Road signs can form very good directional anemometers. They attract very little load if the wind is blowing across the face of the sign, but considerable load if the wind is blowing nearly normal to the face of the sign. In fact, wind tunnel tests (Roy, 1983) have shown that for winds within 30° to the normal to the face of the structure allproduce approximately the same load. This load is very large compared with the drag on the legs, so the effect of the legs can safely be ignored.

The drag on the sign itself can produce large bending moments in the supports. Where the wind velocity is high enough, it can cause bending failure in the supports. The section that has failed in bending can be analysed to determine the load that caused the failure.

Care must be taken in the selection of signs for analysis to ensure that -

(i) the damage was caused by wind alone and not by airborne debris, such as tree branches or building materials. The sign and pole must be carefully examined for wood residue or fresh scratches that may indicate debris impact.

(ii) the damage to the sign must be the formation of a hinge in the upright, not a rotation of the footing in the soil. The failure of an upright can be related to its plastic section modulus, whereas there is little way of quantifying the footing failure.

Figure 5 illustrates the steps in the analysis of the failure of such a sign supported on metal legs.

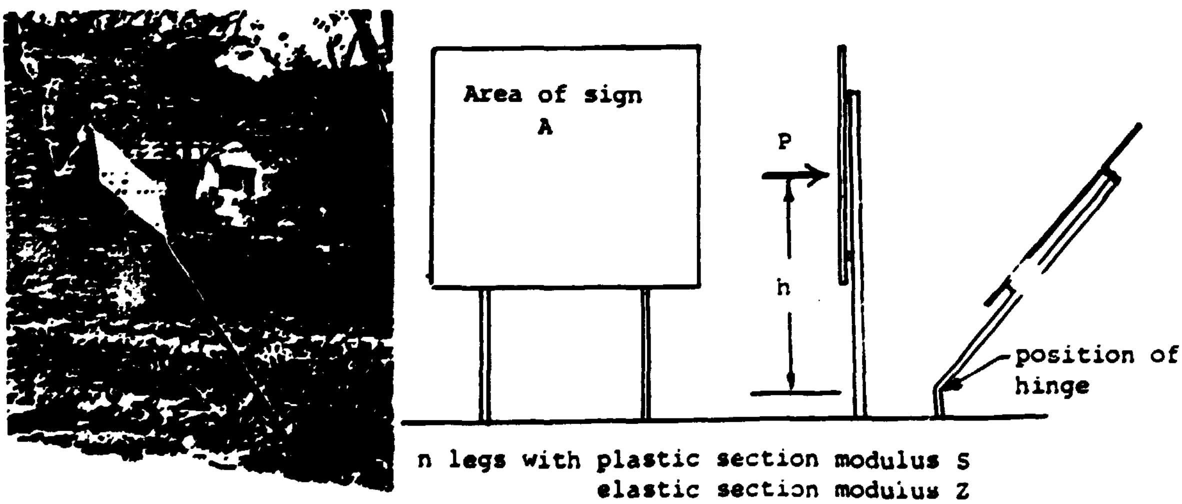


Figure 5 Analysis of Road Signs

This method can be used to analyse signs on one or more vertical legs

n	=	number of legs	
S	=	plastic section modules of each leg	(mm^3)
Z	=	elastic section modules of each leg	(mm^3)
F_y	=	yield stress of leg material	(MPa)
A	=	face area of sign	(mm^2)
h	=	height of centre of sign above position of hinge or potential hinge	(mm)
V	=	maximum gust velocity at height of the sign	(m/s)

(1) Check sign for damage by debris. If there is a possibility of debris damage results will not be conclusive.

(2) Where the sign has been pushed over and hinges have formed in the sign legs, then the moment required to push it over is the plastic moment of the support system.

ie bending moment $> nF_yS$ (Nmm)

(3) This moment can be used to find the load exerted by the wind on the sign.

bending moment = Ph

ie $P > \dfrac{nF_yS}{h}$ (N)

(4) The load exerted by the wind is a function of the wind velocity. The drag coefficient of most signs is 1.2, so

$$P = (1.2 \times 0.6\ V^2)\ A \times 10^{-6} \quad \text{(N)}$$

$$V = 1\ 179 \sqrt{\frac{P}{A}} \quad \text{(m/s)}$$

$$V > 1\ 179 \sqrt{\frac{nF_yS}{hA}} \quad \text{(m/s)}$$

This gives a lower bound on the maximum wind velocity normal to the face of the sign at that location.

The wind velocity had to be greater than that figure at some time to cause the leg failure and blow the sign over.

A converse argument can be used to find an upper bound on the maximum wind velocity. Where a sign had remained upright then we can conclude that the wind normal to the face of the sign was not strong enough to cause failure of the leg.

(2a) Where no damage had been caused to the legs, the material in the leg had not yet become plastic, so the appropriate properly that gives the capacity of the section is its elastic modulus.

ie bending moment $< nF_yZ$ (Nmm)

(3a) is similar to the relationship established above.

$$\text{ie} \quad P < \frac{nF_yZ}{h} \qquad (N)$$

(4a) gives the upper bound on maximum wind velocity

$$V < 1179 \sqrt{\frac{nF_yZ}{hA}} \qquad (m/s)$$

in other words had the velocity at any point in time exceeded V, the sign may have blown over. As it had not, the maximum velocity must have been less than V.

Using these two analyses, every road sign can be regarded as a anemometer.

(B) Fences and Hoardings

These simple structures can also be used for wind speed evaluation using a similar approach to that adopted for the road signs.

Care must be taken to ensure that -

(i) damage was caused by wind alone, not airborne debris

(ii) damage was a bending failure of supports, not rotation of the footing in the ground.

(iii) the hoarding or fence was effectively impermeable, timber palings, bamboo matting or slats may let air pass through and reduce the effective drag coefficient.

In general, only a portion of a fence or hoarding will be blown down, however the aerodynamics of the simple structure is a function of the entire width. Figure 6 presents a typical example with timber supports and the calculations underneath pertain to the evaluation of load. Again the work shown illustrates the principles used in the evaluation of maximum gust wind velocities and is a basic analysis method. Due to the difficulty in defining strength of timber, the estimates will be subject to more variation than for similar structures supported by steel legs.

S_p = spacing between legs in the section which has fallen down (mm)

H,B = cross section dimensions of the timber legs (mm)

F = modulus of rupture for the species of timber used in the leg (dry value) (MPa)

h = height of centre of the hoarding above the fracture in the legs (mm)

h_w = height of the hoarding (top to bottom) (mm)

h'_w = height of the bottom edge of the hoarding above ground level (commonly zero for fences) (mm)

V = maximum gust velocity at the height of the hoarding (m/s)

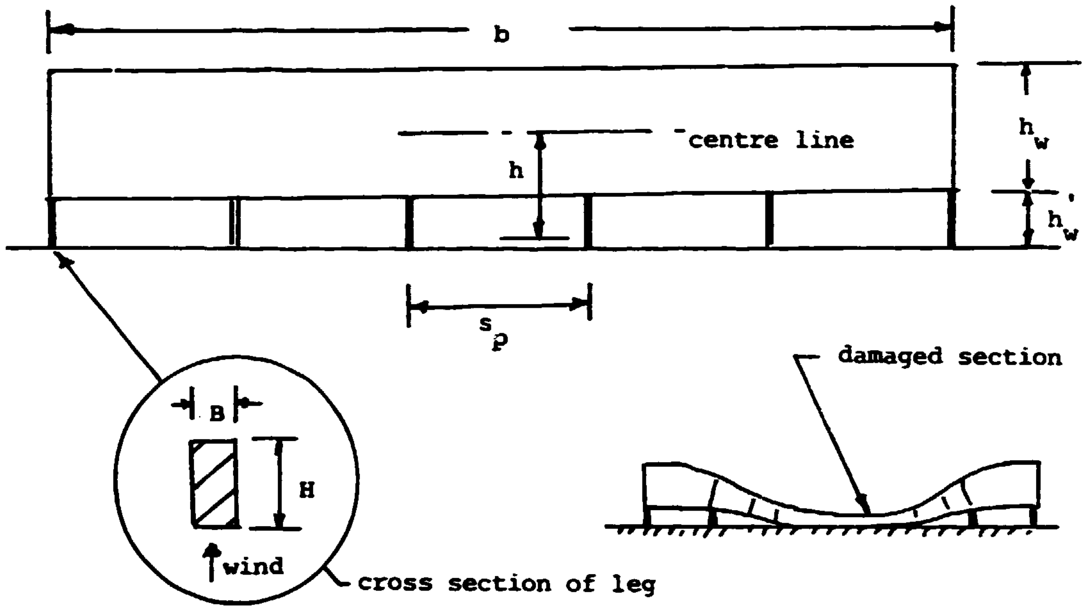


Figure 6 Analysis Of Hoardings and Fences

(1) Check hoarding or fence for signs of recent debris damage. If there is a possibility of debris damage the results will not be conclusive.

(2) Where the timber legs have fractured, the bending moment that caused the fracture can be expressed in terms of the section properties and the modulus of rupture.

$$Z \text{ (elastic modulus)} = \frac{BH^2}{6} \qquad (mm^3)$$

$$\text{then bending moment} > \frac{FBH^2}{6} \qquad (Nmm)$$

(3) This moment can be used to find the maximum load exerted in the centre of the hoarding.

$$P > \frac{FBH^2}{6h} \qquad (N)$$

(4) Now the load exerted by the wind on the hoarding is a function of the wind velocity and the drag coefficient for the hoarding. The drag coefficient is a function of the geometry of the hoarding, in particular, the B/h_w ratio and the value of 'h_w'. Figure 7 gives the drag coefficients (C_D).

$$\text{then} \quad P = C_D \times 0.6V^2 h_w S_p \times 10^{-6} \qquad (N)$$

$$\text{or} \quad V = 1\,290 \sqrt{\frac{P}{h_w S_p}} \qquad (m/s)$$

using the relationship given for P in (3) above

$$V > 530 \sqrt{\frac{FBH^2}{C_D h_w Sp\ h}} \qquad (m/s)$$

Again this produces a lower bound on the maximum wind velocity normal to the face of the hoarding at that location.

By examining hoardings that have not failed, the same expression can be used to give an upper bound, ie the wind cannot have been higher than V, or the hoarding would have failed.

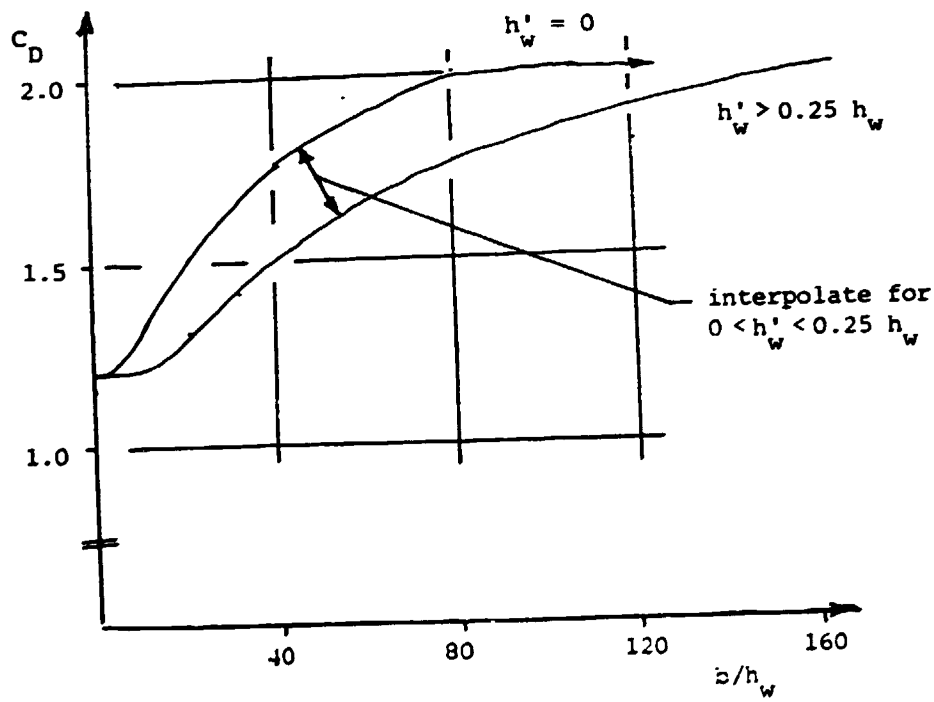


Figure 7 Relationship Between Hoarding Geometry And Drag Coefficient C_D (AS1170 - Part 2, 1983)

(C) Trailer Homes

A third simple structure that can be used to estimate upper and lower bounds on wind velocities is typified by small rigid sheds, caravans or trailer homes. These structures are rarely tied down but rely on their weight to counteract the overturning moment caused by lateral wind forces.

Again prior to the analysis there are some conditions that must be satisfied. These are similar to the conditions for the previous two structures.

(i) damage was caused by wind alone, not by airborne debris.

(ii) the structure was not tied down to the ground in any way but relied on its own weight to resist overturning.

(iii) the structure remained intact and failed by complete overturning. A simplification of the structure to a rigid box is a valid one.

In this case, the weight of the structure must be taken into account in the calculation of V. Uncertainties in the determination of weights of these structures can decrease the reliability of the calculated wind velocities.

Figure 8 shows the salient points used in the calculation of maximum peak gust velocity. Wind tunnel studies have shown that the net overturning moment is independent of angle within 30^{0} to the normal to the structure.

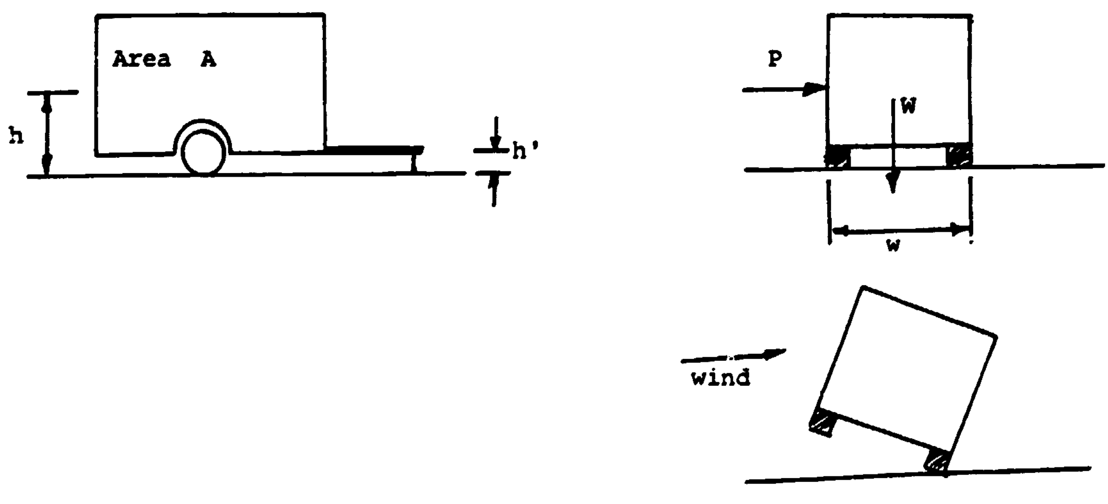


Figure 8 Overturning Of Simple Structures

A = cross sectional area normal to wind (mm^2)
W = weight of structure (N)
h = height to the centroid of windward face (mm)
h' = height of lower edge of structure above ground (mm)
w = wheelbase, or width of structure (mm)
V = maximum gust velocity at the height of the structure (m/s)

(1) Check structure for signs of debris damage or damage prior to overturning.

(2) The restoring moment due to the weight force of the structure is a function of the weight distribution of the structure. Where possible this should be determined by interviewing owners or occupiers.

Assuming an even weight distribution.

$$\text{restoring moment} = \frac{Ww}{2} \quad \text{(Nmm)}$$

(3) For overturning to have taken place

overturning moment > restoring moment

$$\text{ie} \quad Ph > \frac{Ww}{2} \quad \text{(Nmm)}$$

$$\text{or} \quad P > \frac{Ww}{2h} \quad \text{(N)}$$

(4) Drag coefficients can be used to relate wind velocity to the lateral wind force.

For structures with a clear space underneath ($h' > 0$) C_D is approximately 1.1, but for those on ground it can be 0.9. An appropriate value should be chosen.

$$P = C_D \, 0.6V^2 \, A \times 10^{-6} \quad \text{(N)}$$

$$V = 1290 \sqrt{\frac{P}{C_D A}} \quad \text{(m/s)}$$

then using the relationship given for P in (3) above

$$V \quad 912 \sqrt{\frac{W w}{hC_D A}} \quad \text{(m/s)}$$

For structures which have overturned, this gives a lower bound for the maximum gust velocity. The wind speed must have been greater than this to cause the overturning.

If the above relationship is used on structures that have not overturned, it will give an upper bound on the maximum wind gust speed. i.e. if the wind speed had been higher than that figure it would have caused overturning.

2.2. Effects Of Vegetation And Other Structures

During typhoons, brittle or weak vegetation will be removed by the high velocity winds. Shallow rooted trees can also be blown over. However, a number of more resilient bushes and trees can resist the force of the wind and serve to break up the wind stream. Strong structures can also cause significant surface roughness that can influence wind velocities downwind of them.

2.2.1 Boundary Layers

The wind at a great height above the earth's surface has a very high velocity. There is a decrease in velocity of the wind as the earth's surface is approached. This is due to the drag of the wind along the surface. When expressed in this way, it becomes obvious that the rough surfaces have a larger drag effect than smooth surfaces.

Typical velocity profiles are shown in Figure 9. Various mathematical models for this distribution have been put forward to represent the velocity gradient. The complexity of some of these is probably not justified by the accuracy of measurements of wind speed from which they have been derived.

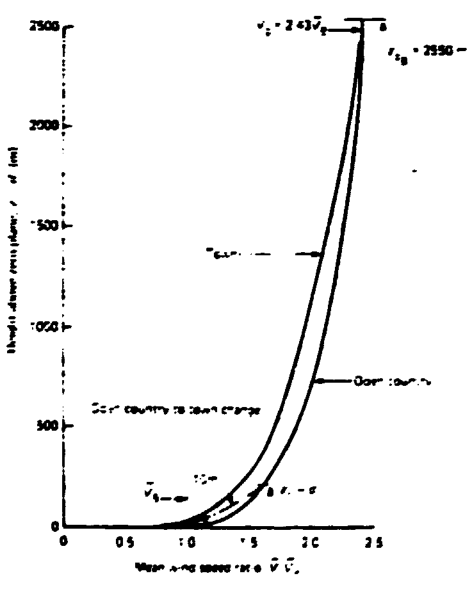


Figure 9 Variation Of Velocity With Height

Most wind codes in use throughout the world incorporate models of these distributions, however a word of caution needs to be sounded before indiscriminantly using them.

Some codes such as the current British and American Codes use average wind speeds and the height/windspeed relationships for average wind speeds are very different to the equivalent relationships for peak gust velocity. In subsection 2.1.2 a number of methods were presented that produced estimates or bounds on peak gust velocities. Peak gust records can also be very easily accessed from anemometer records. The boundary layer representations given in the British and American Codes are not suitable for direct operation on those wind speeds.

In section 2.4 the maximum gust wind speed will be used at the height of buildings that are being assessed to calculate wind loads at the time of damage. Again height/speed relationships appropriate to peak gust data are used for those operations.

For these reasons the boundary layer representation presented in the Australian Wind Code will be used in this monograph. The Australian standard uses only data appropriate to the peak gust velocity.

2.2.2 Australian Standard Terrain Categories

The roughness of the earth's surface is very difficult to quantify. The code categorises terrain based on the vegetation and type of structures that covers the surface. Four categories are defined in the Australian Standard.

PLEASE NOTE THAT WHILE THEIR NAMES ARE SIMILAR TO THOSE USED IN THE BRITISH CODE, THESE CATEGORIES DO NOT FOLLOW THE SAME CONVENTION.

TERRAIN CATEGORY 1 - Exposed terrain with few or no obstructions and in which the average height of any objects surrounding the structure is less than 1.5m.

This includes water bodies, flat treeless plains (there are a few of these in Australia), large expanses of rice paddy where there are no trees.

TERRAIN CATEGORY 2 - Open terrain with well scattered obstructions having heights generally 1.5 to 10m.

This includes land in many rural settings, open parkland, the outskirts of towns and villages, and in the case of cyclones, buildings located in bushland.

TERRAIN CATEGORY 3 - Terrain with numberous closely spaced obstructions having the size of domestic houses.

This terrain category has been specifically established for towns and villages. Dense resilient bushland may also be classified as this category provided the trees have proper ability to withstand typhoon winds without extensive defoliation or loss of limbs.

TERRAIN CATEGORY 4 - City centres with many tall buildings or large industrial complexes.

Most structures examined in assessment programmes are less than 6m high, and for this type of structure it can be considered as completely embedded in its terrain if it is more than fifty metres from the edge of the terrain. This means that only the houses within fifty metres of the edge of town and villages need be considered as having a terrain Category 2 approach direction. The others could be classed as being embedded completely in terrain Category 3.

For each case in which an assessment is made or a structure analysed to determine wind speed, the terrain immediately upwind of the structure should be assessed on the basis of the above data.

$$V_S = K_Z \ V_R$$

V_S = peak gust velocity at structure height (m/s)
K_Z = multiplier (function of height and terrain category)
V_R = regional peak gust velocity

K_Z has been tabulated in the Australian Standard and that table is reproduced here as Table 1. For heights between 0 and 15 metres the same data has been plotted in Figure 10.

VARIATION OF DESIGN WIND VELOCITY WITH TERRAIN AND HEIGHT

Height z m	Velocity multipliers			
	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
Up to 3	1·00	0·90	0·65	0·65
5	1·02	0·93	0·65	0·65
10	1·09	1·00	0·70	0·65
15	1·12	1·03	0·75	0·65
20	1·13	1·06	0·85	0·70
50	1·21	1·15	1·00	0·85
100	1·27	1·22	1·11	0·98
150	1·30	1·27	1·18	1·06
200	1·33	1·30	1·23	1·13
250	1·35	1·33	1·27	1·17
300	1·35	1·35	1·30	1·22
350	1·35	1·35	1·33	1·26
400	1·35	1·35	1·35	1·30
450	1·35	1·35	1·35	1·32
500	1·35	1·35	1·35	1·35
Above 500	1·35	1·35	1·35	1·35

Table 1

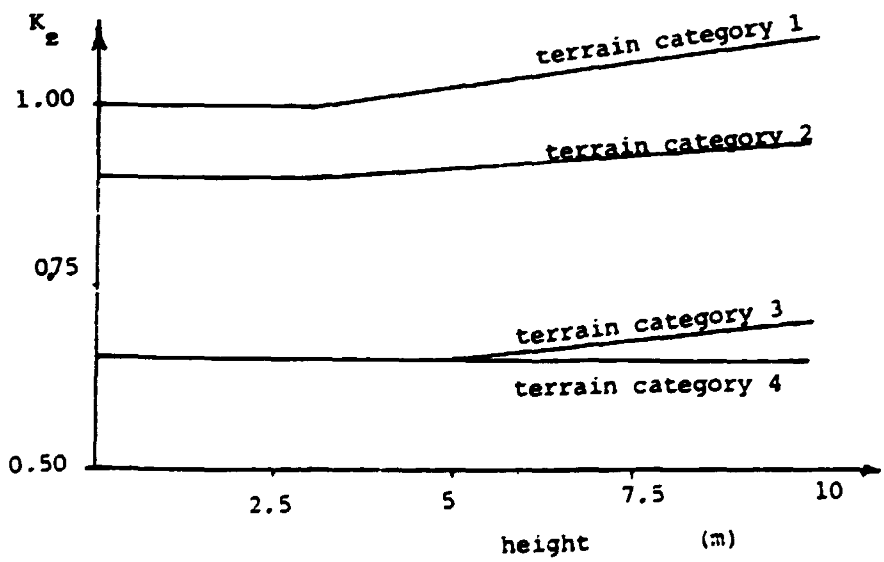


Figure 10 Variation Of Velocity With Terrain And Height

It is to be noted that all velocities given here are peak gust velocities as they are the ones that do the damage. If an average speed is required for the regional wind velocity to perform an extrapolation of the type described in Section 2.1.1 then the standardised peak regional velocity could be divided by 1.6 to give an average regional velocity at 10m height in terrain category 2.

2.3 Effects Of Variations In Topography

As the wind passes over the earth's surface it is affected by changes in shape of the earth's surface. Hills, ridges, cliffs and valleys all cause reductions of wind velocity at some points and increases in wind velocity at other points. This section attempts to quantify those changes so that the wind velocity calculated for any given structure can take due account of all the effects of the site.

2.3.1 Qualitative Examination Of Topographical Effects

(i) When the wind flows directly up a gently sloping ridge as shown in Figure 11(a), the flow near to the ground de-accelerates near the base of the escarpment, as the flowlines curve upward (V_b). The flow lines at this point are fairly widely separated. In contrast, near the top of the slope the flow lines are more crowded and the flow accelerates (V_c). Once over the crest, however, the flow lines spread out causing de-acceleration, with the speed finally returning to a value close to the initial approach velocity (V_d).

When the wind flows directly down the gently sloping ridge, the flow pattern is almost identical to that for flow up the ridge, but with the flow direction appropriate to that shown in Figure 11(a).

When the wind flows directly up a steeply sloping ridge, as shown in Figure 11(b), separation occurs near the base of the slope and eddies are generated immediately against the slope at its foot. This pushes the de-acceleration zone (V_b) well away from the foot. Reattachment of the flow generally occurs near the top of the slope. At the crest, separation can occur again. The eddies generated at the top can mean that structures just behind the crest of ridges can be sheltered. The acceleration zone (V_c) is generally just downslope of the crest, and the de-acceleration generally takes place well away from the crest (V_d).

Flow down a steep ascarpment differs from the flow up the escarpment due to the non-linearities in the flow caused by the separation of flow. The effect is illustrated in Figure 11(c).

Where the flow is at an angle to the ridge, the component of flow parallel to the ridge is unchanged, but that normal to it is changed as described above.

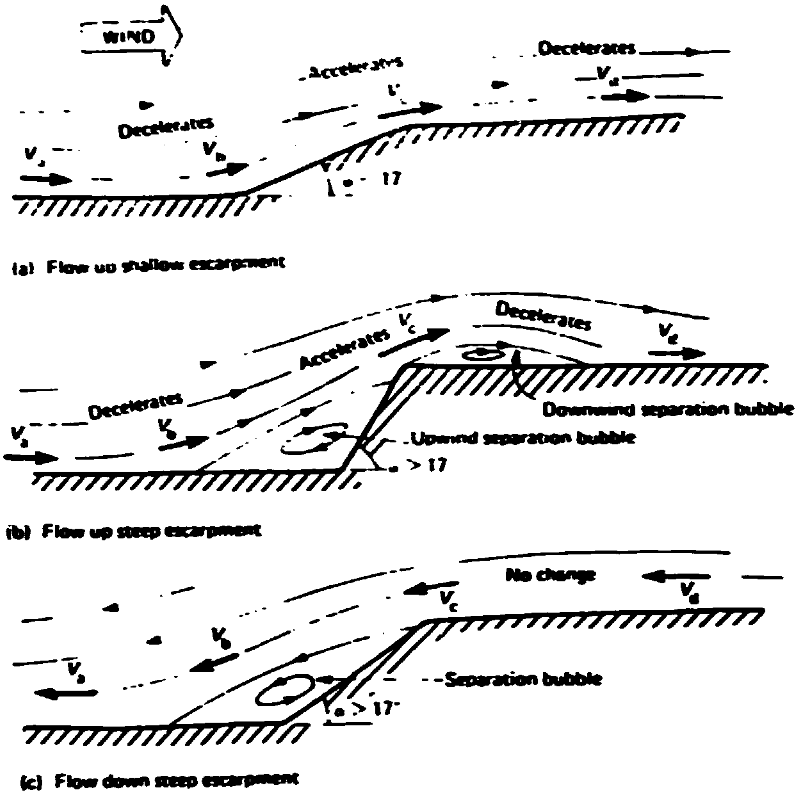


Figure 11 Flow Over Escarpments from Cook (1985)

(ii) Hills

A hill differs from a ridge in that wind may flow to either side. The flow on the centre line of the hill remains much the same as the ridge representation, but much of the flow moves outwards to pass on either side of the hill and then inwards again afterwards. Thus not only the magnitude of the wind velocity, but also its direction is changed.

Separation and reattachment may occur for very steep hills. On a hill, the acceleration of the wind is a maximum at the crest, but it is generally less than the acceleration for a comparable ridge.

(iii) Valleys

Where the wind is blowing directly up a valley or at a slight angle to it, the valley tends to direct the flow along its axis. If the valley is open at the windward end, there will be some acceleration of the flow due to the tunnelling effect.

Where the wind is blowing across the line of the valley and the valley is steep sided, it remains fairly protected because a separation bubble is formed between the valley walls. Where the valley is shallow or very wide it behaves much like a gently falling excarpment followed by a gently rising ridge. It can be simulated by the effects shown in Figure 11(a).

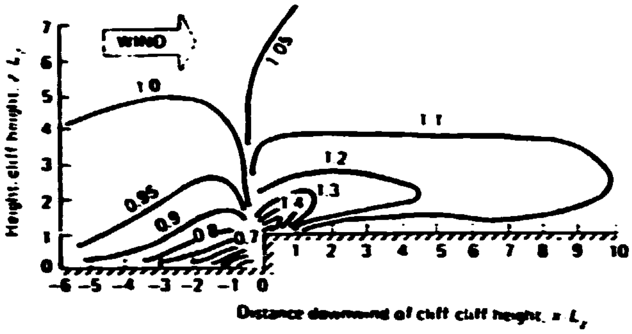
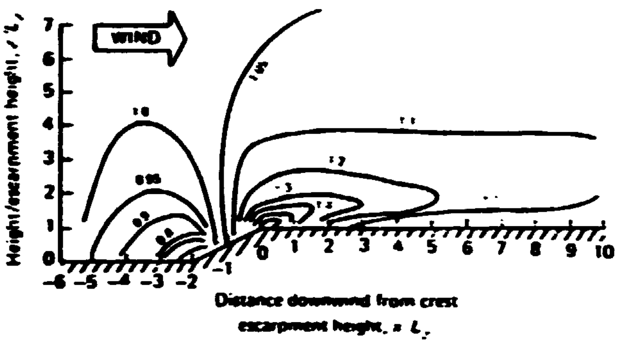
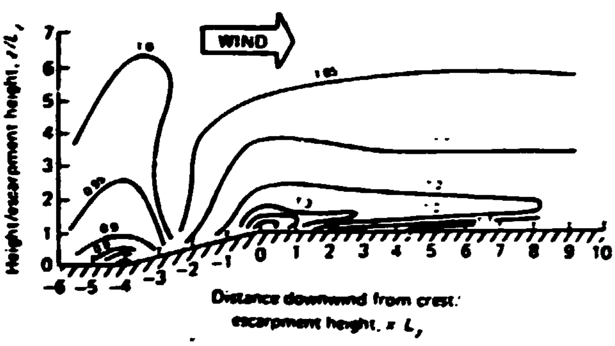


Figure 7.10 Speed-up ratio for cliffs (from reference 111)

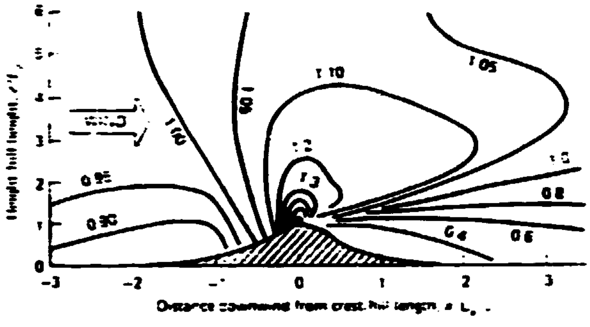


(a) Steep slope, ψ = 0.50

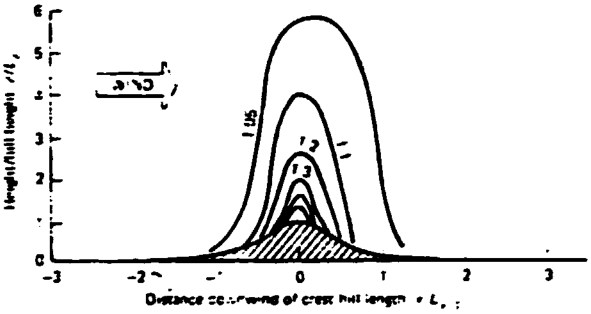


(b) Shallow slope, ψ = 0.25

Speed-up ratio for sharp-edged escarpments (from reference 111)



(a) Steep slope



(b) Shallow slope

Figure 12 Speed-up Ratio For Topographic Effects

2.3.2 Modification Factors For Topography

The general form of the modifcation is

$$V = K_t V_s$$

V = expected velocity at structure site and height (m/s)

K_t = speed-up ratio for topographic effects

V_s = velocity at structure height as computed in Section 2.2.2 (m/s)

The relationship given above can be used directly where the wind is at right angles to the topographical effect. Values of K_t are given in Figure 12 which has been taken from work performed on topographic effects in New Zealand.

Where the wind direction is not normal to the topographic feature, some modification to the velocity at the structure site must be made. The method is illustrated in Figure 13 and is presented below.

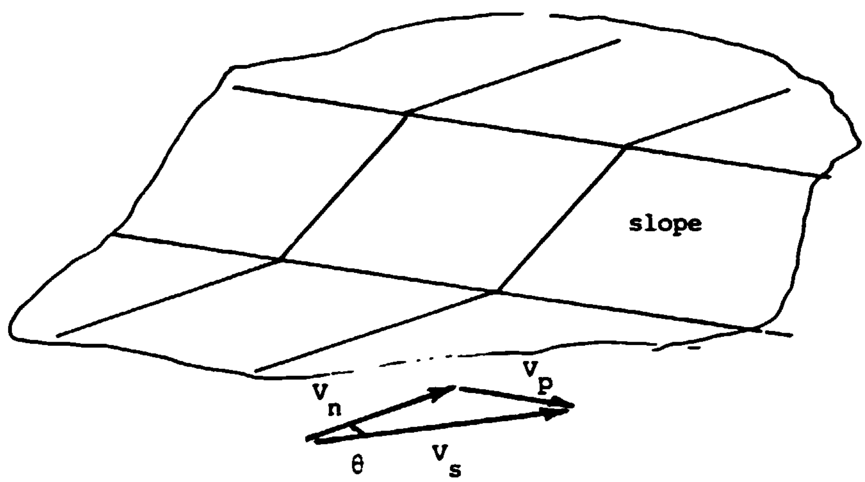


Figure 13 Wind At An Angle To A Topographic Feature

With the wind at an angle to the topographic feature, the structure height velocity can be resolved into two components, one normal to the topographic features (V_n) and the other parallel to the topographic feature (V_p).

$$V_n = V_s \cos \theta$$

$$V_p = V_s \sin \theta$$

The speed-up effect does not apply to the parallel component of the velocity but only to the normal component. The wind velocity at the structure height is the vector sum of the two components.

ie $V = \sqrt{(V_n . K_t)^2 + V_p^2}$

The angle that the modified wind makes to the slope will be changed slightly by the change in speed of the normal component.

2.3.3 Use Of Topographic And Terrain Modification Factors

There are two main uses to which these factors are put.

(A) Conversion Of Peak Gust Wind Speeds At Simple Structures To Regional Wind Speeds

Using the velocity estimation procedures outlined in Section 2.1 the velocity at a particular structure may be determined. In order to relate the wind speed to other structures which may have quite different settings, the regional wind velocity must be found.

(i) site structure windspeed is modified to account for topographic effects. This produces the structure height wind speed.

$$V_s = \frac{V}{K_t}$$

(ii) structure height wind speed is modified to account for structure height and terrain

$$V_R = \frac{Vs}{K_z}$$

This produces the regional peak gust wind velocity for a particular location. That velocity can be used for other structures in that location or it can be extrapolated to other positions.

(B) Conversion of regional wind speed to peak gust velocity at structure level.

Once a regional wind speed has been established for an area it can be used to find the wind loads on buildings at specific sites in that area. To do this the wind speed must be modified to account for terrain, structure height and topography.

(i) Determination of peak gust regional wind velocity.

If the regional wind velocity determined for a particular area has been based on average wind speed data it must be modified to be a peak gust regional windspeed.

$$V_R = \overline{V}_R \times 1.6$$

$\overline{V}_R$ = mean hourly regional wind speed

V_R = peak gust regional wind speed

(ii) Modification to account for terrain and structure height

$$V_s = V_R K_z$$

This produces the windspeed at the structure height for the appropriate terrain.

(iii) Modification to account for topographic effects.

$$V = V_s K_t$$

This produces the actual expected peak gust velocity at the structure height and can be used to find the expected wind loads on the structure.

2.4 Wind Loads On Buildings

This subsection presents a method for obtaining the loads on buildings from a given structure height wind speed. For structures that are not enclosed, such as masts, lattice towers or industrial facilities, drag coefficients should be drawn from standard fluid mechanics texts.

2.4.1 Wind Pressure

Wind is simply a fluid (air) moving. As such it obeys the basic laws of fluid dynamics. The most fundamental of these is Benoulli's theorem which quantifies the fact that if a moving fluid is slowed down, its pressure must increase. Houses, along with other obstructions on the earth's surface contribute to drag which effectively slows down the air near the ground. The pressure in the immediate vicinity of these obstructions must increase.

A free stream of air moving at velocity V has the capability of mobilising the following pressure P

$$q = \frac{0.6V^2}{1000} \qquad (kN/m^2)$$

This pressure is known as the free stream dynamic pressure. It should be evaluated for each building at roof height.

2.4.2 Pressures On Buildings

The actual pressure on any building surface can be related to the free stream dynamic pressure by pressure coefficient. Negative pressure coefficients indicate suction normal to the surface.

For any face of the building

$$p = C_p q \qquad (kN/m^2)$$

Then for any face of a rectangular building as shown in Figure 14, the appropriate values of Cp can be used to find pressures on all faces. Note that the wind direction relative to the building axis is given as an angle Θ. Prior to the determination of the wind loads, it is necessary to infer from either the damage itself or an interview with the occupants, the direction from which the wind was blowing when the damage was initiated.

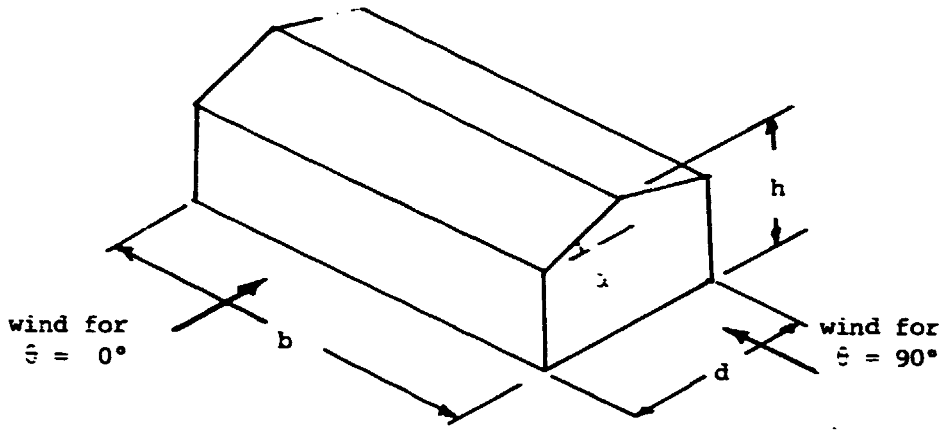


Figure 14 Notation For Pressure Coefficients On Buildings

For wind in any direction the external pressure coefficients for any wall can be found using the following data.

(i) Windward Wall Surface

	Buildings set on the ground	$C_p = +0.6$
	Buildings set above the ground ie open underneath	$C_p = +0.8$

(ii) Side Walls Surfaces

	These are the walls that the wind blows parallel to	$C_p = -0.6$

(iii) Leeward Wall Surface

	All buildings with roof slope $< 25^o$	$C_p = -0.3$
	Where $\Theta = 0^o$ and roof slope $> 25^o$	$C_p = -0.5$
	Where $\Theta = 90^o$ and roof slope $< 25^o$	$C_p = -0.3$

For all buildings with a roof slope of less than 10^o, the uplift pressure on the external surface of the roof is given by the following data.

Distance from the windward leading edge of the roof	C_p
0 to h	-0.9 (uplift)
h to 2h	-0.5
2h to 3h	-0.3
more than 3h	-0.1

The above figures can also be used for a building with a roof slope of more than 10^o if the critical wind was blowing from $\Theta = 90^o$.

If the wind was blowing such that $\theta = 0^{o}$ then external pressure coefficients from the following table must be used.

EXTERNAL PRESSURE COEFFICIENTS C_p FOR ROOFS OF BULDINGS FOR $\theta = 0°$ WITH $\alpha \geq 10°$

h/d	Windward slope angle α, degrees								Leeward slope angle α, degrees		
	10	15	20	25	30	35	45	≥ 60	10	15	≥ 20
≤ 0.25	−0.7	−0.4	−0.3, +0.2	−0.2, +0.3	−0.2, +0.3	+0.4	+0.5	+0.01α	−0.2	−0.5	−0.6
0.5	−0.9	−0.6	−0.4	−0.3, +0.2	−0.2, +0.2	−0.2, +0.3	+0.4	+0.01α	−0.5	−0.5	−0.6
≥ 1.0	−1.3	−1.0	−0.7	−0.5	−0.3, +0.2	−0.2, +0.2	+0.3	−0.01α	−0.5	−0.6	−0.6

The pressure coefficients given above can be used to find a rough estimate of the total load on the building. For each wall surface the external pressure, p, is found from

$$p = C_p q_z \qquad (kN/m^2)$$

The load on each surface can be found by mulitplying the pressure by the external area of the surface.

Where cladding has been damaged, it is necessary to calculate internal pressure coefficients as the net load on the cladding is a combination of external pressure and internal pressure.

2.4.3 Cladding Loads

Loads on the cladding are a function of the external pressures and the internal pressures. However, as the cladding attracts external load over very small areas, turbulance effects can cause very localised high suction on both walls and roof. Over the large surface area of the whole building these loads are effectively damped out, but must be taken into account in the determination of the local cladding loads.

(i) Local Pressure Factors In Suction Areas

Over quite small areas very large suctions can be mobilised on both roof and wall surfaces. The Australian Standard defines a dimension, a, over which the local pressures may act. Figure 15 illustrates the definition of a.

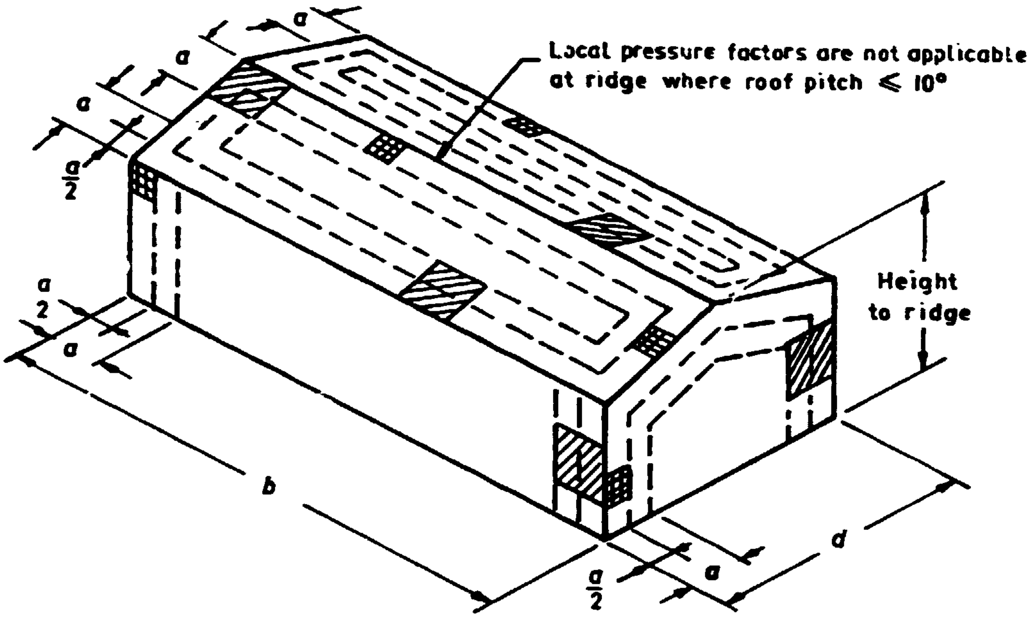
a is the smaller dimension of h
or 0.2 b
or 0.2 d

On any external cladding surface normally subjected to suction the peak suction could be increased due to local turbulence effects as follows -

Over an area of a x a the suction could be increased by a factor of 1.5

Over an area of 0.5 a x 0.5 a the suction could be increased by a factor of 2 .0.

Where damage to cladding has been observed, these two effects should be evaluated and the one which places the damaged element under the highest load should be used for calculating the cladding load.



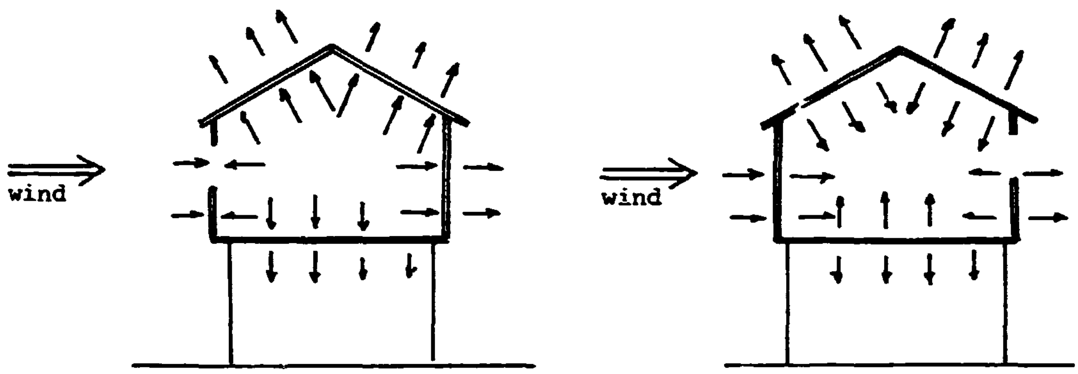
a = the height to ridge or 0.2*b* or 0.2*d*, whichever is the least.

Figure 15 Local Pressure Zones

(ii) Internal Pressures

The position of openings in a building can have very significant effects on the cladding loads. Figure 16 illustrates the effect with a dominant opening in just one wall. In Figure 16(a) the dominant opening is on the windward wall and it admits the positive windward wall pressure to the internal air space. With this configuration of wall openings, there is little or no net pressure differential on the windward wall, but a very large net pressure on surfaces with external suctions such as the side and leeward walls and the roof.

In Figure 16(b), the dominant opening is on the leeward wall, and it reduces the pressure differential across that wall. This produces an internal suction. The internal suction causes a large net pressure on the windward wall and reduces the net pressure on all other surfaces.



(a) dominant opening in windward wall

(b) dominant opening in leeward wall

Figure 16 Effect Of Openings On Internal Pressures

Often in reality, the configuration of internal openings is not as straight-forward as Figure 16 would lead us to believe. Unless buildings are completely lined, there is often an air space around the top of walls under the eaves. Many buildings with raised floors have air leakage through the floor itself and there is also leakage around nominally closed windows and doors. As most of the surfaces of houses are leaky and most are under suction, the net effect of the general leakiness is to reduce internal pressures.

Dominant openings are seldom designed into buildings. Damage to windows and doors on the windward side of the building due to wind presure or airborne debris may cause a rapid increase in internal pressure. The presence of positive internal pressures may double the net load on cladding elements, so it is critical that internal pressures be correctly evaluated.

Advice from the occupants of the building will be particularly useful in this regard, particularly if they were in the building during the passage of the typhoon.

(1) As accurately as possible, determine which windows and doors were open or damaged at the time of the first serious structural damage to the building.

(2) Estimate the porosity of the floor, walls (especially around the top) and roof. This may be expressed either as dimensions of the clear opening e.g. 100mm gap at the top of a 1.7m wall, or as a percentage e.g. 5% porosity of the floor.

(3) This will enable the area of opening on each wall to be assessed. A permeability ratio can be found as.

$$r_p = \frac{\text{total area of opening on windward wall}}{\text{total area of opening on all suction surfaces}}$$

If $r_p > 1$, then the internal pressure coefficient is given by the following relationship and is positive.

r_p	C_{pi}
1.0	+0.1 or -0.1
1.5	+0.3
2.0	+0.5
3.0	+0.6
6 or more	+0.8

If $r_p < 1$, then the internal pressure coefficient is negative.

r_p	C_{pi}
1.0	-0.1 or +0.1
0.8	-0.3
0.5	-0.4
0.3 or less	-0.6

Care taken with this part of the load determination often pays dividends, particularly where there is significant cladding damage to the building.

The determination of the loads on elements that have failed is particularly important in making any damage assessment useful for the correction of errors in building design or construction practice. It is often one of the least exciting aspects of damage investigation but it is a very important part of it. At the site investigation stage, it does not take much extra time to record the information that will make load determination possible.

3.0 STRUCTURAL ACTION OF HOUSING AND SMALL BUILDINGS

The previous sections have concentrated on general consideration such as the planning for a typhoon damage assessment and the calculation of wind speeds for an area damaged by strong winds. In this chapter, more emphasis will be placed on individual buildings, their structural behaviour and assessment of their performance under the loading of strong winds.

It is to be emphasised that this is not a design exercise; the house under examination will have a structural configuration with which we must work, and the wind gust that caused the damage will have come from a single direction. The assessment process is one of analysis in which the structure and the wind loading are known and the technician must establish which of the many failures evident on the damaged building was the first one to occur.

If necessary, a design process may be adopted later to select an appropriate improved detail to prevent a similar failure from occurring in the future.

The first step is to gain an understanding of the way in which houses resist wind loads, as it is quite different to the way in which larger buildings carry loads.

3.1 Houses Without Structural Cladding

These types of houses are common in all parts of the world and include timber framed houses clad with weatherboard, timber framed brick veneer houses, framed houses with bamboo or cogun grass matting as wall cladding.

In many cases the wall material may be structural, but a roof cladding that is not able to carry in-plane loads, may be used. Steel sheeting, asbestos or fibre-cement sheets or plywood sheeting roofs can all mobilise resistance to in-plane forces, but thatch, tiles or shingles cannot carry in-plane forces. The comments in this section with reference to the roof structure are applicable to the roofs of houses clad with thatch, tiles or shingles.

3.1.1 Transmission Of Lateral Loads

The cladding can attract out-of-plane loads due to the pressures evaluated in the previous sections and these loads must be carried by bending to the structural frame of the house. The structural frame must then carry the wind loads safely to the ground. A summary of the actions is illustrated in Figure 17 and is presented below for horizontal loads on the wall cladding.

(i) windloads on cladding carried by bending in the cladding to vertical frame members (studs).

(ii) studs carry loads by bending to top and bottom horizontal members (wall plates).

(iii) wall plates carry loads by bending to the top of diagonal braces which in turn, carry the load to ground.

(iv) diagonal braces act in tension.

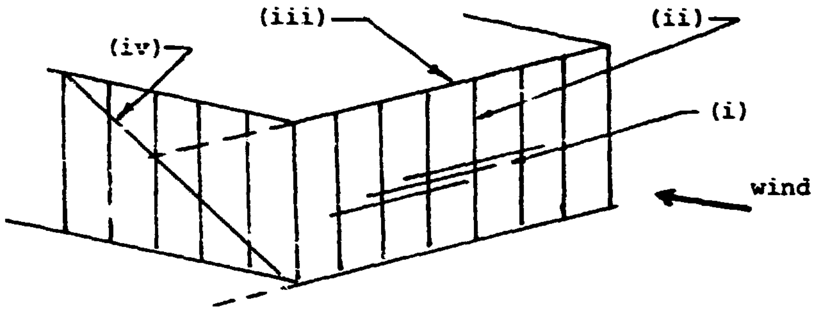


Figure 17 Load Transfer Through Frame

The diagonal members mentioned in (iv) above are generally incorporated in walls running normal to the windward wall of the house as shown in Figure 17. Where the houses are set above the ground, the bottom plate maybe supported by diagonal bracing between the legs.

3.1.2 Evaluation Of Member Loads For Lateral Forces

Most structural systems in which the cladding does not carry in-plane loads can be regarded as statically determinate.

(A) Studs

Once the pressure on the wall is known it can be used to find the load on each stud. Studs span as simply supported beams between top and bottom wall plates. The bending moment and reactions at top and bottom can be found by simple statics.

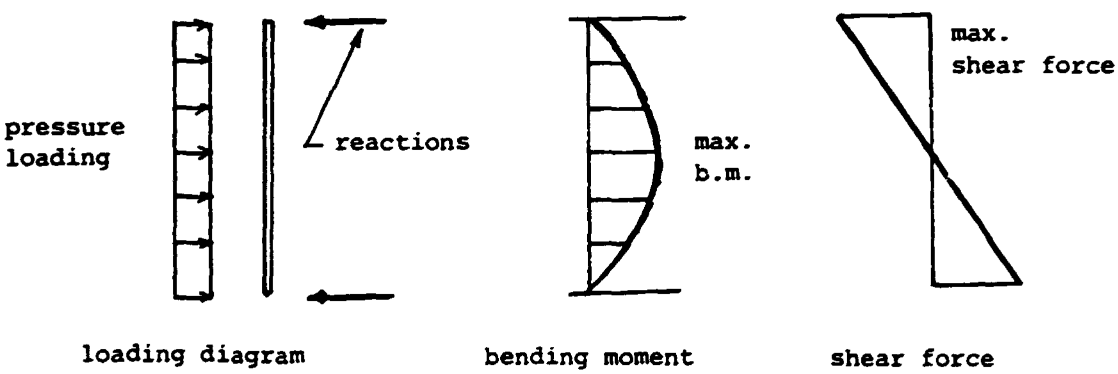


Figure 18 Loads On Studs And Stud Connections

Maximum bending moment $= \dfrac{p\ s\ H^2}{8}$ (kNm)

Reaction at top and bottom $= \dfrac{p\ s\ H}{2}$ (kN)

p = net pressure across wall cladding (kN/m^2)
s = spacing of studs (m)
H = height of studs (m)

It can be seen that the maximum bending moment is in the centre of the stud. If the stud were to fail in bending under the action of wind loads, the failure would be expected near mid-height.

Likewise the connection at the stud/wall plate junction must be capable of carrying the maximum shear force in the stud.

(B) Wall Plates

Wall plates, particularly the top wall plate, can be regarded as spanning between transverse walls. They are located directly by the top of the studs. This load is applied at discreet points along the plate, but within the accuracy of the other approximations in the analysis of the action of the house, it can be regarded as a uniformly distributed load.

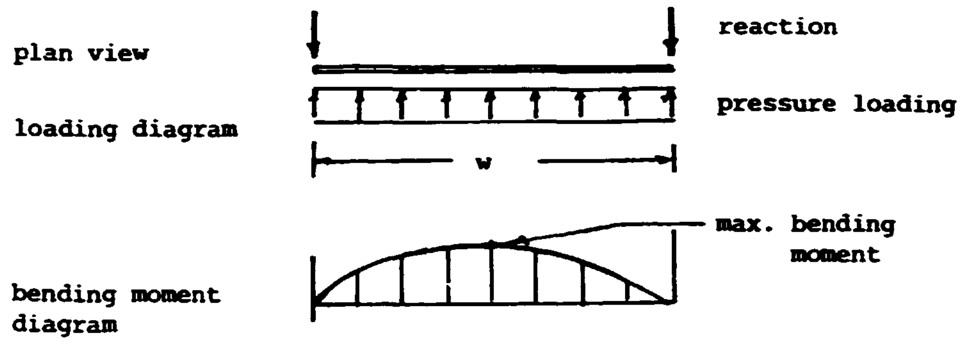


Figure 19 Loads On Wall Plates And Wall Plate Connections

W = width of room ie distance between transverse walls
H = height of studs
p = net pressure across wall cladding

effective uniformity distributed load $= \dfrac{p\,H}{2}$ (kN/m)

maximum bending moment $= \dfrac{p\,H\,W^2}{16}$ (kNm)

reaction at each end $= \dfrac{p\,H\,W}{4}$

Again, the maximum bending moment occurs at midspan of the wall plate, so if bending failure occurs, it would be expected in the centre of a room. An interesting diversion is that the maximum bending moment can be very large for large W. This makes wall failure in big rooms much more likely than in small rooms. As a result shelter during a typhoon should be taken in the smallest rooms in the house to minimise the risk of injury due to wall failure.

(C) Transverse Walls

These walls are loaded at the top by the wall plates as indicated above. They must then carry the horizontal loads to the bottom of the wall by in-plane strength. When cladding cannot provide this, a brace is generally used. Where the cladding cannot provide in-plane resistance and no diagonal brace has been fitted to the wall, it is generally ignored as a structural component of the house for carrying lateral loads.

The transverse walls can attract lateral load from each side of the house. Pressures on the windward wall give reactions at the top of transverse walls in the same direction as the suctions in the leeward walls.

The transverse walls must therefore be analysed for lateral loads at the top equal to the sum of all reactions at that wall on the windward wall plus the sum of all reactions at that wall on the leeward wall.

Figure 20 shows the salient points in the analysis of the structural behaviour of the wall.

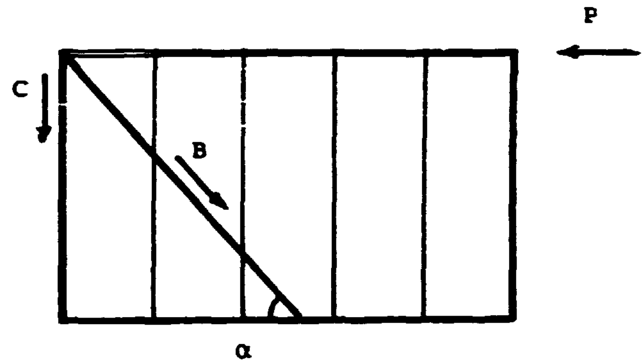


Figure 20 Loads On Transverse Walls

The strength of the transverse wall is due to the fact that it has formed a triangulated structure with the brace and a vertical stud carrying axial forces.

P = Sum of all reactions at bracing walls for both windward and leeward top plates (kN)

B = Axial force in diagonal brace (kN)

C = Axial force in wall stud (kN)

α = Angle that diagonal brace makes with the horizontal

then $B = \dfrac{P}{\cos\alpha}$ (kN)

and $C = B \sin\alpha$

$C = P \tan\alpha$ (kN)

Where a brace is tied into a number of studs and is constructed of a reasonably stiff member such as a length of timber or bamboo, it can also carry compression if the wind was blowing from the other direction. However, steel strap, wire, or rope braces can only be regarded as tension members and their compression strength must be ignored.

Where diagonal braces are used between the legs of a high house, the same analysis is used to find the loads in the members.

Generally the bracing members themselves can carry those axial forces safely, but often the connections do not have sufficient strength to carry the loads. Bracing failures most commonly originate at the connections.

3.1.3 Transmission Of Uplift Loads

As was shown in Chapter 2, the roof is generally subjected to uplift forces. Frequently those forces are well in excess of the weight force of the roofing and roof structure, and can result in pieces of the roof or roof structure being torn free from the rest of the house.

Generally the transmission of the uplift loads within the roof structure is independent of the in-plane strength of the roof or wall cladding.

A number of siimplifying assumptions can be made with regard to the transmission of the uplift load within the roof. These primarily relate to load distribution between the various structural elements within the roof.

(i) Cladding loads are carried to cladding fasteners in direct proportion to the area of cladding supported by each fastener. Near edges of the roof, local pressure factors indicated in Section 2.4.3(i) should be incorporated in the external pressures, and internal pressures should be calculated in accordance with section 2.4.3(ii). The weight of roofing material can be significant, particularly where the roofing is clay or concrete tiles, or wet thatch.

(ii) Cladding loads from the fasteners are directly transmitted to the roof battens. While these members are long and are therefore generally continuously spanning across their supports (the rafters), the loads at each roofing fastener are generally well correlated. It is therefore valid to assume that the battens receive a uniformly distributed load from the roofing. The edge battens can be loaded by the areas of roofing subjected to the local pressure factors, so where the full span of a batten falls within one local pressure area, the local pressure factor should be used to determine the loads on those battens.

(iii) The batten loads are carried to trusses or rafters that generally span across the building. The span of those members is generally too large for the local pressure factors to be used for their loads. The rafter loads generally take into account the external average pressure, the internal pressure and the weight of roofing and battens. Batten spacing is often close enough for the assumption of uniform loading to be a valid one.

(iv) The rafter or trusses are generally tied into the top of the walls. Where very heavy roofing is used, the weight of the roof alone may be sufficient to counter the aerodynamic uplift loads. The force on the truss or rafter anchorage points can be calculated directly from the load on the rafters.

(v) Particularly where lightweight roofing and wall cladding is used, the uplift loads may be higher than the total structure weight, and the complete building may have to be tied into the ground. Anchorage to the ground is achieved by incorporating structural members in the walls that can carry the tensile forces associated with uplift right through to the footings. Loads on the footings are also affected by the tension and compressive forces due to the overturning moment of the lateral loads on the complete building.

Recognition of all these effects is necessary in the calculation of the uplift loads on elements within the structure. Many elements that are stressed primarily by uplift forces are also required to carry loads due to the lateral forces.

3.1.4 Evaluation Of Member Loads For Uplift Forces

In this case, even though the structural members in the roof structure are not statically determinate, their low flexibility and the high correlation of wind loads on adjacent elements means that contributory areas can be readily calculated.

(A) Roofing

Nearly all roofing systems are tied down to battens and the roofing to batten anchorage presents a possible weakness in the elements resisting uplift in a building. The area of roofing that is supported by a single fastener can be taken as a rectangle with side lengths equal to the batten spacing and the roof fastener spacing as shown in Figure 21. Local pressure multipliers must be used for fasteners around the edge of the roofing.

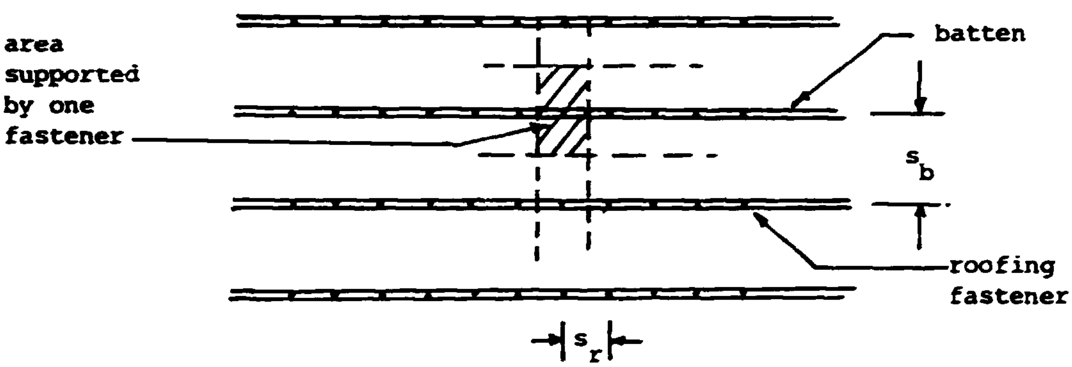


Figure 21 Area Of Influence - Roofing Fasteners

p_e	=	average external suction on roofinng	(kN/m^2)
p_i	=	internal pressure on roofing	(kN/m^2)
l_f	=	local pressure factor at edges	
t	=	thickness of roofing	(M)
w	=	unit weight of roofing	(kN/m^3)
s_r	=	spacing of roofing fasteners	(m)
s_b	=	batten spacing	(m)
α	=	roof slope	

Net uplift pressure $= (l_f \cdot P_e) + p_i - w.t.\cos\alpha$ (kN/m²)

load/fastener $= s_r \cdot s_b \cdot [l_f \cdot P_e + p_i - w.t.\cos\alpha]$ (kN)

w is an approximate density - low values are conservative

saturated thatch	w	=	8 kN/m³
dry thatch	w	=	3 kN/m³
profiled steel sheets	w	=	80 kN/m³
concrete or clay tiles	w	=	20 kN/m³

Generally the highest values of p_e are for fasteners near the upwind edge of the roofing. At this location l_f will be either 1.5 or 2 depending on the area of roofing supported. The highest unit fastener loads are at that point.

(B) Battens

The relationships for loads on battens are very similar for those on roofing fasteners. Battens are generally fairly light members whose weight can safely be ignored. They carry load primarily in flexure.

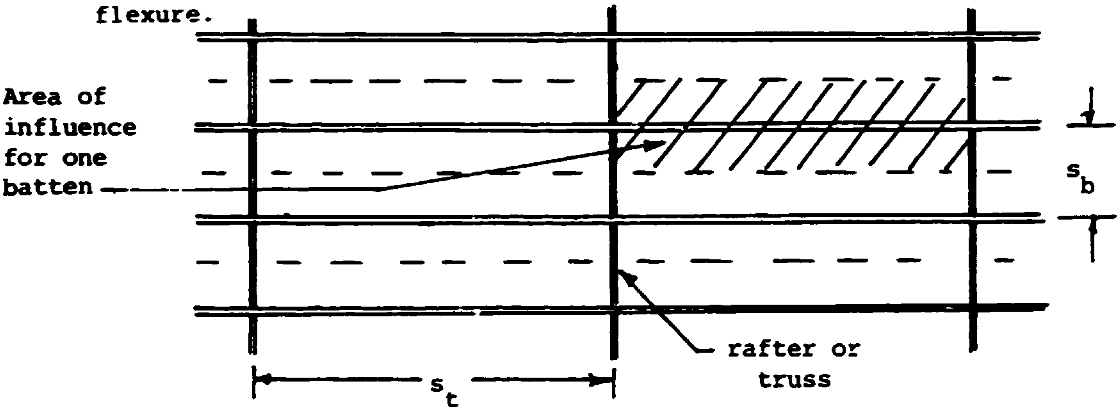


Figure 22 Area Of Influence - Battens

The net uplift pressure is the same as that for the roofing fasteners shown above and can be used to find the maximum bending moment in the batten.

s_b = batten spacing (m)

s_t = rafter or truss spacing (m)

Maximum bending moments can be calculated for continuous battens or in battens near butt joints or at the end of the building where negative moments cannot be carried over rafters.

Maximum continuous batten moment $= \dfrac{s_b s_t^2}{12} (l_f \cdot P_e + P_i - w.t\cos\alpha)$ (kNm)

Maximum discontinuous batten moment $= \dfrac{s_b s_t^2}{8} (l_f \cdot P_e + P_i - w.t\cos\alpha)$ (kNm)

Maximum force in batten to rafter connection

$$= \frac{s_b s_t}{2} \; (l_f . p_e + p_i - w.t \cos\alpha) \qquad \text{(kN)}$$

Again the moments and forces in battens tend to be higher, closer to the edge of the building.

(C) Trusses Or Rafters

Generally the complexities of trusses mean that they have to be analysed individually. No generally applicable formula for their analysis can be presented. Fortunately however, their structural action is sufficiently reliable to prevent significant damage in most cases.

(D) Truss Anchorage

For some of the lighter roofing materials, the weight of roof trusses can be comparable with that of the roofing itself. With the heavier roofing, the trusses have to be stronger to carry gravity loads, so are frequently heavier as well. A reasonably reliable "rule of thumb" is that the unit weight of the roofing can be doubled to allow for the weight of the roof structure in calculation of uplift on the roof structure anchorage.

b = width of the house (span of trusses) (m)
s_t = spacing of trusses or rafters (m)

Maximum truss anchorage force

$$= \frac{s_t b}{2} \; (l_f . p_e + p_i - 2\, w.t \cos\alpha) \qquad \text{(kN)}$$

3.1.5 Interaction Of Lateral And Uplift Forces

Many of the members in houses in which the cladding materials do not have in-plane strength or stiffness carry either uplift or lateral loads in isolation. There are only two main cases in which the two may overlap.

(A) Wall Studs

In resisting lateral loads, the wall studs act in bending and must transfer lateral reactions at each end to the wall plates. In resisting uplift loads wall studs can be used to carry vertical tension force from the truss anchorages to ground. Under those conditions, the members themselves are in tension and the connections must also carry tension.

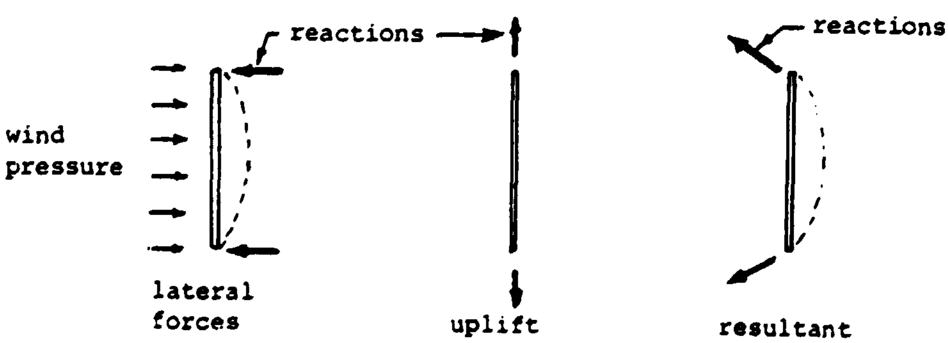


Figure 23 Combined Lateral And Uplift Forces on Stud

Under combined tension and flexure, timber and many other fibrous materials have brittle failure characteristcs, particularly if dry.

The connections under these circumstances must be able to transmit the vector sum of the horizontal reaction and the truss anchorage load.

(B) Sub-floor Structure Legs

Where diagonal bracing is used below floor level, the legs may be subjected to axial forces in order to carry the overturning moment due to the lateral forces on the house. The tensile axial forces will directly add to the uplift forces induced by the action of wind on the roof.

In evaluating the net tension force in the legs due to uplift, the weight of the entire house and contents must be subtracted from the aerodynamic uplift.

3.2 Houses With Structural Cladding

In many respects the analysis of this type of house is more complex because it can be structurally indeterminate. In some parts of the world this type of house predominates yet in other parts it is virtually unknown. Mud brick walling, wattle and daub walls, plywood walls, fibre cement sheeting cladding and even plasterboard linings are all capable of resisting in-plane forces and hence can all be regarded as structural wall claddings.

Fibre cement roofing, steel roofing, or plywood roof or ceiling linings can all give the roof structure an inherent strength in resisting in-plane loads. This strength and stiffness has little effect on the structural mechanics of carrying uplift loads as detailed in Section 3.1.3 and 3.1.4, but it has significance in the resistance of lateral loads.

3.2.1 Transmission Of Lateral Loads

Again, lateral loads are attracted by the cladding which generally spans vertically to carry the loads to the roof structure and to the floor. In the case of thick mud walls, the high stiffness and horizontal arching action in the bricks may allow the walls to span horizontally directly to adjacent walls.

Houses that have thin wall cladding materials generally incorporate a framework that supports the cladding on vertical studs. The studs are able to carry lateral loads by bending to roof and floor planes, as shown in Figure 17.

Generally there is a load redistribution by elements in the plane of the roof that transfers the lateral loads to transverse walls. The structural mechanics of that redistribution will be discussed later in this subsection. Once the redistribution has taken place, the loads are carried as in-plane forces through the transverse walls. In subsection 3.1.1, the only load path available was through a diagonal brace, but for the materials under consideration in this subsection, shear transfer within the cladding may be possible.

The shear transfer within the cladding of transverse walls implies that the cladding is securely fastened to the wall framing. Generally where failures have been observed in the transverse walls, they have been due to an overloading of the fastening system used to join the cladding to the frame.

Where the buildings incorporate a roofing material that can carry in-plane forces such as steel sheeting, plywood or fibre-cement boards, the lateral load transfer is effected by diaphragm action within the roof. The principle of roof sheeting load transfer is illustrated in Figure 24.

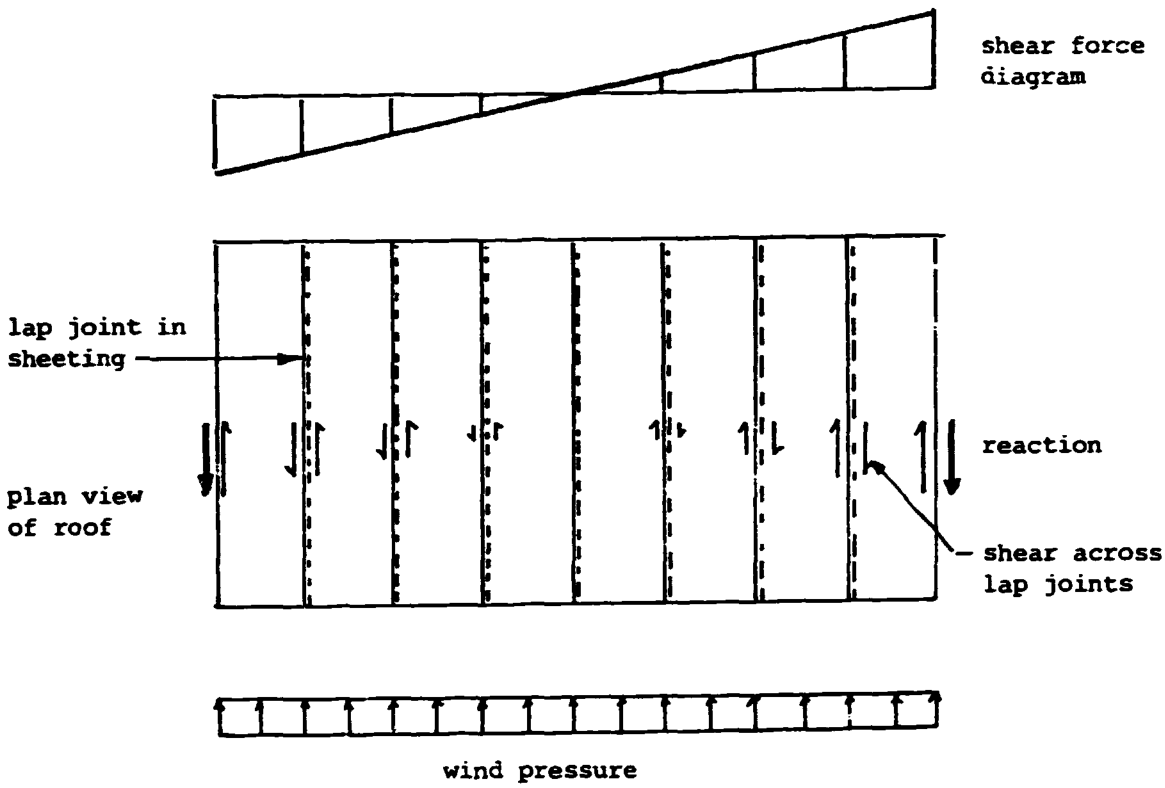


Figure 24 Diaphragm Action In Roof Sheeting

The roof sheeting functions as a very thin but deep beam and carries load from the centre of each room sideways to the transverse walls. This thin, deep beam has zones of shear weakness and flexibility at regular intervals throughout its length, where the shear must be transferred across the lap joints between individual sheets. At the lap joints the shear must be transferred by fasteners that penetrate both of the sheets at the lap.

Generally for buildings the size of most houses, the fastening systems used to anchor the sheeting to the roof structure also have sufficient strength and stiffness to effect the required shear transfer. The stiffness of the roof diaphragm means that it can carry load past some internal walls and on to others. This makes the lateral load distribution within a house quite complex. An approximate method for the load distribution between the transverse walls is outlined in the next subsection

3.2.2 Evaluation Of Member Loads For Lateral Forces

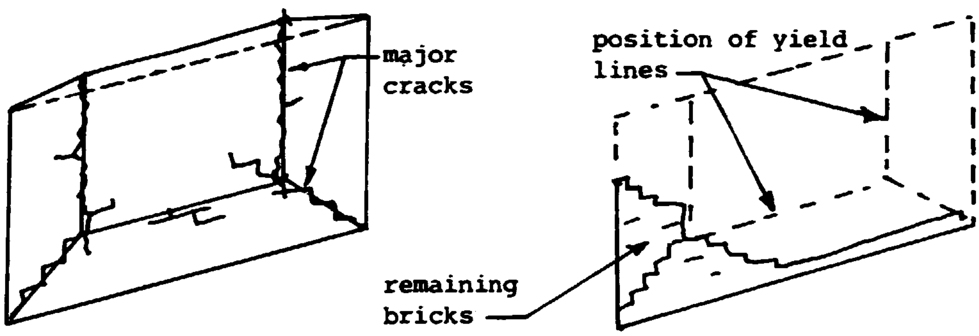
(A) Thin cladding systems fastened to frames.

The analysis of the loads on the framing members is identical to that outlined in subsection 3.1.2(A) for studs.

(B) Thick, stiff cladding systems.

This type of system is restricted to solid brick, stone or mud brick walling. The stiffness of these types of walls is much higher than that of the roof structure. Their failure tends to be brittle and the behaviour is similar to a concrete slab subjected to out-of-plane loads. A rigorous analysis of the wall system could be achieved using a yield line method.

The failure of such wall systems due to wind pressure lends itself to yield line analysis because the yield lines can be inferred from the damage pattern. In many cases the collapse will not be total so the yield lines can be plotted readily onto a sketch of the wall as indicated in Figure 25.



(a) wall damaged but standing (b) wall partially collapsed

Figure 25 Typical Yield Line Positions For Brick Walls

Where the brickwork has cracked but not collapsed, the yield lines can easily be identified but where the brickwork has partially collapsed, the break lines can actually be below the yield lines as shown in Figure 25. Careful examination of the debris will be required to identify the position of the yield lines.

Yield line analysis will produce a moment capacity of the brickwork at failure. These calculations are very useful as very little is known about the actual performance of many types of brickwork under out-of-plane typhoon wind loads.

Reinforced concrete design texts cover the fundamentals of yield line analysis.

(C) Lateral Load Transfer Through Roof Sheeting

Except where a house has only two structural transverse walls, one at each end, the stiff roof diaphragm can function as a continuous beam. The total lateral force on the house that is carried to the roof must be apportioned to the transverse walls for transmission to ground.

Tests performed on full scale houses (Boughton, 1987b) have shown that the load is apportioned according to the in-plane stiffness of the transverse walls. For most cladding systems the stiffness of the walls is proportional to the length of the wall that is free of openings.

The total lateral load on the house can be found from the work presented in Chapter 2. This load has a contribution from pressure on the windward wall and suction on the leeward wall. The load transmitted to the roof is half of the total lateral load on the building as the remainder is carried by the studs to the floor level.

The total opening-free length of transverse wall in the house can be found as shown in Figure 26. It is to be noted that only walls clad with materials capable of transmitting in-plane loads need be considered in this calculation.

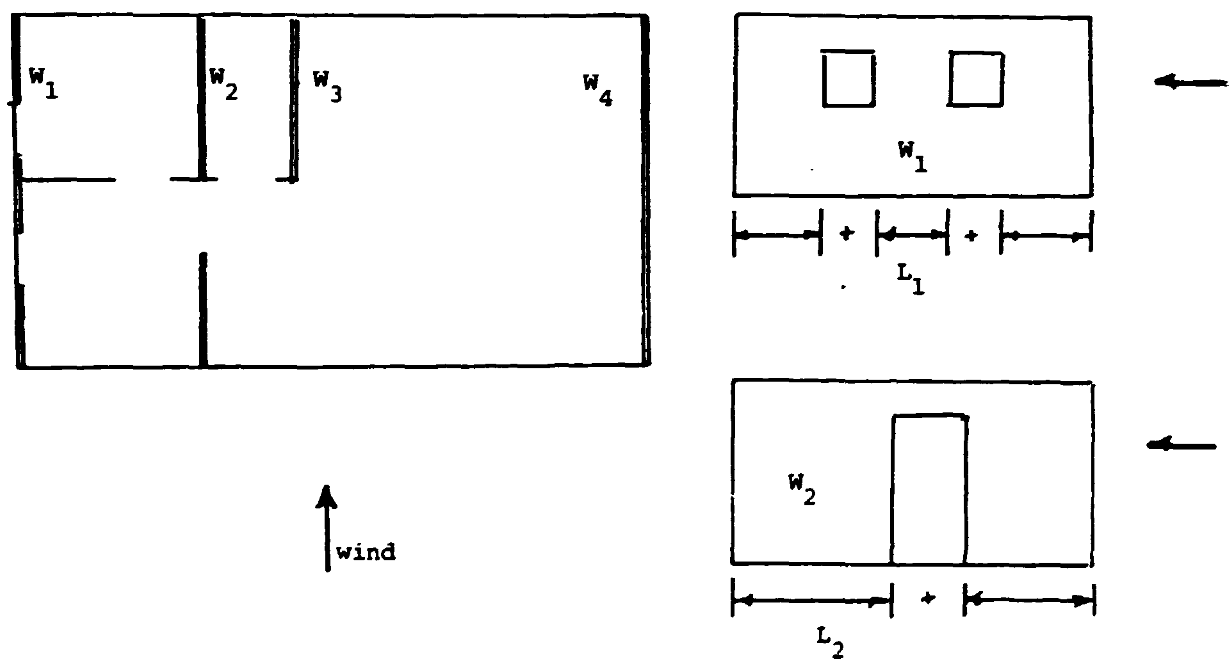


Figure 26 Length Of Potential Bracing Walls

The length of potential bracing walls does not include that part of the wall under or over doors or windows.

$$\Sigma L_b = L_1 + L_2 + L_3 + \ldots\ldots L_n$$

ΣL_b = sum of potential bracing walls for the house (m)

L_1 = length of wall W_1 free of openings (m)

n = number of transverse walls in the house

then the lateral load carried by wall W_1 is given by the following expression

$$P_1 = \frac{L_1}{\Sigma L_b} P_T \qquad \text{(kN)}$$

P_T = total lateral load on the building (kN)

P_1 = lateral load carried by in-plane forces in wall 1 (kN)

These calculations can be repeated for as many walls as necessary in order to evaluate the performance of the building.

Once the lateral force distribution within the house has been derived the shear force diagram for the roof can be drawn. This will enable the shear carried across lap joints to be evaluated. Methods for drawing shear force diagrams are generally indicated in elementary strength of materials texts.

3.3 Damage Observation And Types Of Failure

The quantitative methods outlined in the previous two subsection enable the evaluation of the load carried by damaged building elements. It is the determination of loads that enables damage observations to be turned into building assessments.

3.3.1 Accuracy And Number Of Observations

Often the information required to calculate the loads can be obtained by using estimated dimensions, as the accuracy of the wind speed data is often not very high. There will be a large scatter of the results which reflects the poor accoracy of the load calculation and also the large scatter often observed in the behaviour of building materials even under well controlled laboratory conditions. If enough assessment of the same type of damage are made, then the large number of observations will improve the confidence that can be attached to the conclusions in spite of the scatter.

Where possible, at least six assessments of the same type of damage should be made. A daily appraisal of the number of buildings checked should be made to ensure that effort is not wasted by reporting in great detail the same type of damage. Unless there is a particular reason to concentrate effort on one particular damage pattern, no more than twenty buildings with identical damage patterns should be reported in detail. This does not preclude a cursory examination of more buildings.

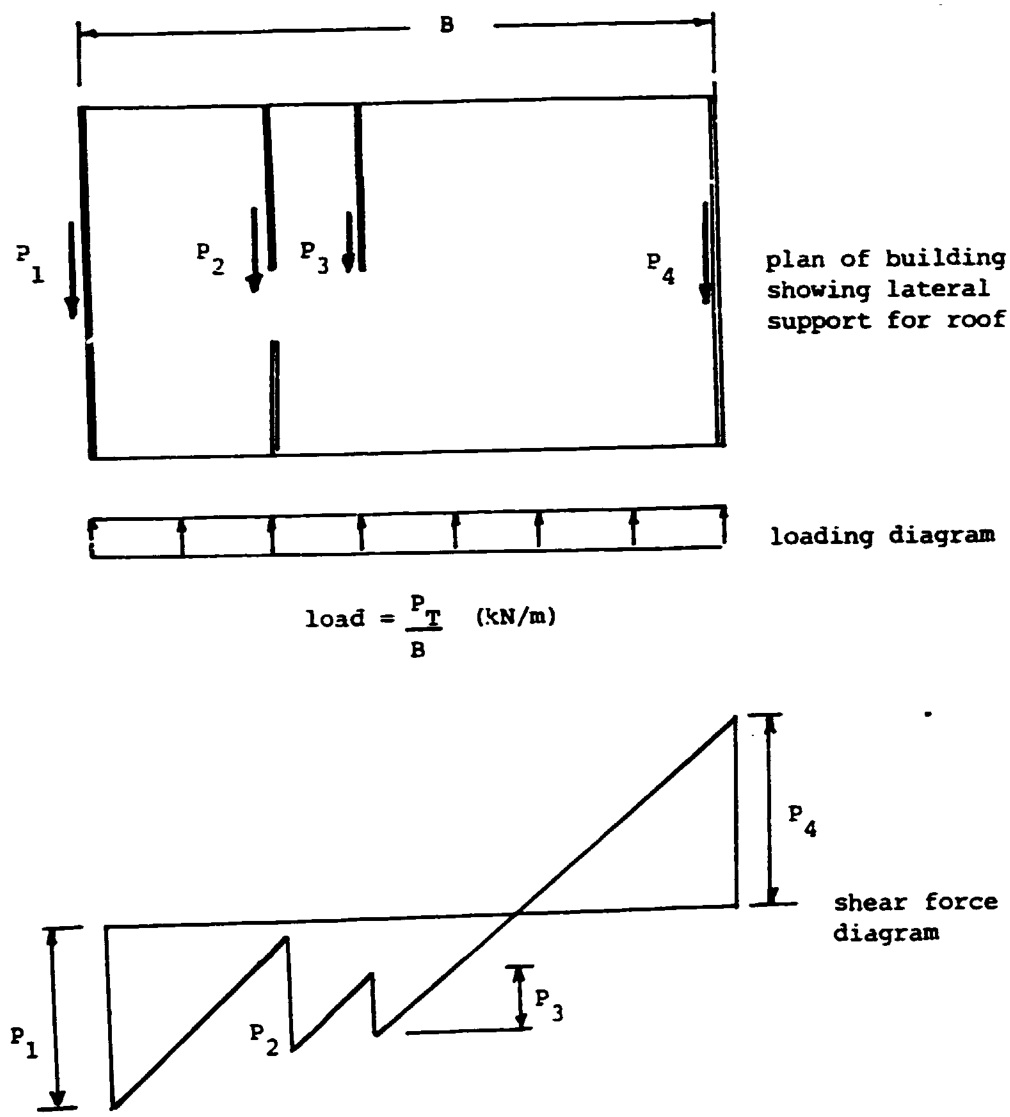
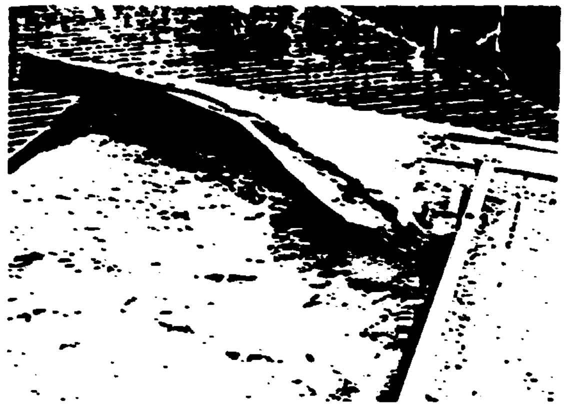


Figure 27 Shearforce Diagrams For Roof

3.3.2 Observation Of Damage

The scale of damage due to the passage of a typhoon can be highly variable in any location. Some buildings may be totally flattened, while others may have little visible structural damage. Often discrepancies in damage may occur within one street. Damage types can be classified as follows and shown in Figure 28.

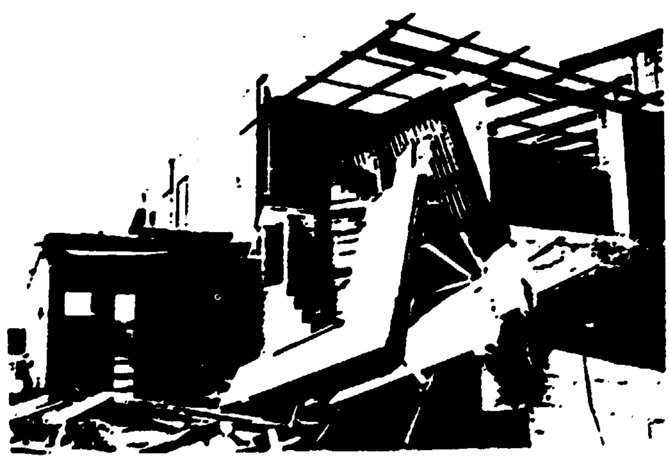
superficial damage



simple structural damage

local structural damage

severe structural system damage



very severe damage

Figure 28 Damage Types

(i) Superficial Damage

This type of damage is usually confined to non-structural elements in a building. It may include sandblasting, or damage to roof gutters, window treatments or contents.

(ii) Simple Structural Damage

This type of damage is structural failure but is confined to a single type of structural element. It may include failure of roof fasteners, batten fastening or lateral bracing. The distinguishing feature of simple structural damage is that damage is confined to a single feature.

For example, all the batten connections in a house may have failed, but if all other details are intact, including the roofing fasteners, then the damage is simple structural damage.

(iii) Local Structural Damage

This type of damage is structural, but is confined to one area of the building although many structural elements may have failed. Typical examples would be damage to one corner of the roof in which roofing connections, batten connections and perhaps battens had all failed. The damage within the one house was not widespread and all the matching components could be readily identified.

(iv) Severe Structural System Damage

In this type of damage many different types of elements may have failed over a wide area of the building, but they are all part of a single structural system. For example, widespread roof damage in which many components cannot be located and which incorporates damage to roofing, various connections and failure of some battens and rafters would be classed as severe structural system damage, because all the damage was confined to the roof system and was due primarily to the effects of uplift loads.

(v) Very Severe Damage

This type of damage usually renders buildings irrepairable. Extensive damage to both roof structure and walls makes the effects of lateral and uplift loads impossible to separate.

Very severe damage can make damage observation extremely complicated most of the failures would have occurred as a secondary effect during the collapse of the building. It is very important to identify the first elements to have failed and to differentiate between those initial failures and subsequent damage. There are a number of techniques that can be used to find the first elements to have failed.

(A) Examination Of Similar Partially Damaged Houses

Frequently it is very helpful to examine partially damaged houses before examining badly damaged examples of the same type of building. Very different performance of identical buildings can be caused by different conditions of environmental shelter. Buildings in hollows, behind ridges or protected by dense resilient vegetation can have simple structural damage whereas identical structures in open locations can sustain very severe damage.

The examination of buildings with simple structural damage can facilitate the location of the weakest elements in the structure. The very severely damage buildings can be checked to locate the subtle signs of first failure.

(B) Search For Signs Of First Damage

A number of subtle signs can be used to locate the first failures in a severely damaged building. The fine scale of these signs means that often a small hand lense must be used to verify them. In houses with very severe damage it must be possible to identify particular elements as having come from one particular house. It also helps to have a prior knowledge of sources of weakness within the structure. This can be gained by examination of similar buildings showing simple structural damage.

(C) Retrospective Analysis

This is the most complex technique at our disposal to isolate first failures. Some failures may be found among the debris that are simple to analyse. These may include flexural failure of clear timber or a ductile tensile failure of steel. The failure loads can be determined from the geometry of the components and material properties. These can be used with the wind loads on the structure to infer the condition of the structure when these failures took place. By repeating this exercise a number of times an illustration of the partially damaged house can be built up. This picture can often then be reinterpreted as a simple, or at least, local failure and hence lead to the components involved in the first failure.

3.3.3 Commonly Observed Damage Mechanisms

Through damage assessment programmes, a number of damage patterns become obvious. Some of the more common ones are described briefly below in order to facilitate their identification.

(i) Fatigue Of Roofing

The prolonged uplift on the roofing through the duration of a typhoon is accompanied by movement of the separation bubble causing the uplift. This can produce a large number of load/unload cycles. Over the hours during which this cycle is repeated, fatigue cracks may develop in metal sheet roofing. These occur in the regions of highest stress - immediately next to the fasteners.

The fatigue cracks start at the holes made by the fasteners and work slowly outwards. Once the crack has gone past the head of the fastener, the fastener can pull through the sheeting.

The loss of a single fastener causes the load carried by adjacent fasteners to be increased. This extra load speeds the crack propagation process and can lead to accelerated failure of those fasteners.

Generally the roofing fasteners carying the highest loads are near the edge of the sheeting and the failure of a number of connections causes the roofing to lift. This, in turn, changes the aerodynamics of the roof and increases the overall load on the sheeting. The aerodynamic effect and the load redistribution effect can combine to increase the fastener loads in the remaining connection to well in excess of this failure load. At this stage the roofing over a large area of the building may be torn free.

The loss of the roofing can have secondary implications. With no roofing to keep out the rain the rest of the house can become rapidly saturated. Some wall and/or ceiling claddings can lose strength when wet and allow failure of the house due to lateral loads. In other cases where the roof sheeting was effecting the distribution of lateral loads to transverse walls, the loss of the roof can lead to collapse of the windward wall.

This example can also be used to illustrate the types of damage discussed in the previous section:

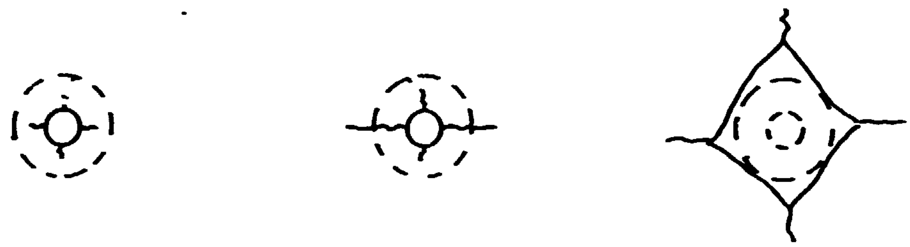
Fatigue failures can require a large number of load/unload cycles to be initiated. They can often occur late in the period of high winds and hence when loads are falling. On some buildings, a number of fasteners may have failed but the loads may have then been low enough to prevent widespread propagation of the failure. This would be a simple structural failure.

In other cases, the flapping roof sheeting may cause damage to the batten system and after a few roofing fasteners had failed, battens may break in bending before the remaining roof fasteners had been overloaded. This would produce local structural damage.

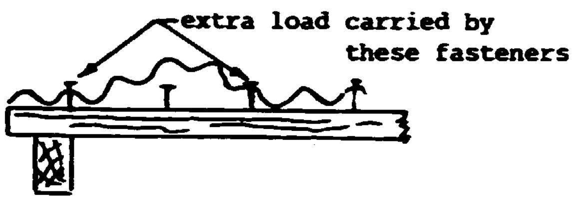
Where the roof sheeting and/or part of the roof structure had been torn away and disintegrated, but the remainder of the house had remained intact then the damage can be regarded as severe structural system damage.

If, on the other hand, the loss of the roofing had caused walls to collapse as well then the damage could be classed as very severe damage.

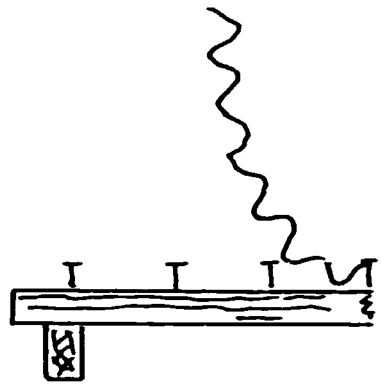
The various processes in the damage described above are illustrated in Figure 29.



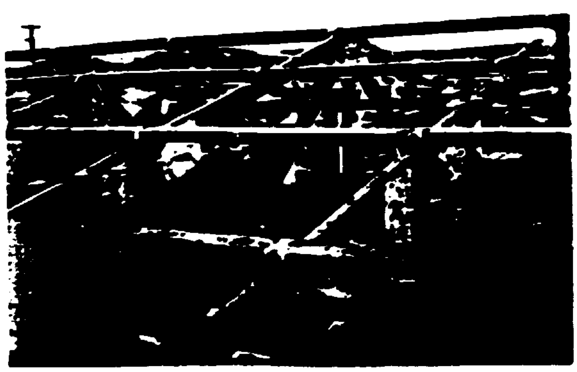
crack initiation crack propagation fastener pull through



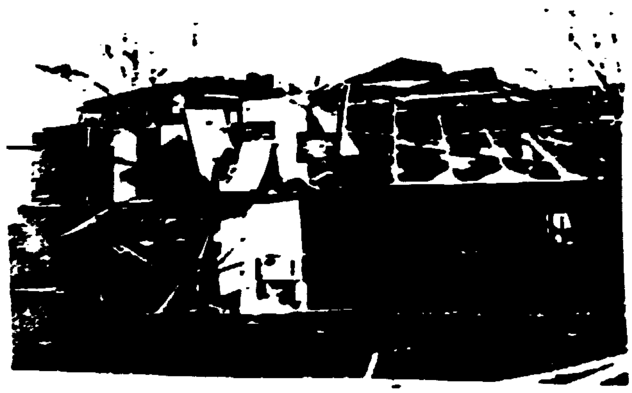
load redistribution



sheeting lifting



total loss of sheeting



wall damage following roof loss

Figure 29 Fatigue Of Roofing - Progressive Damage

(ii) Inadequate Strength Of Specific Elements

Generally a number of identical connections are repeated throughout the building and if the connection used does not have adequate strength to carry the wind loads, then widespread failure results. This can be the case with batten fasteners or truss anchorages.

Again it is the long duration of typhoon winds that causes propagation of failure through the structure. Where short duration wind loads such as those associated with isolated storms cause failures, the damage is generally restricted to one or two elements. However, during typhoons the structure has to resist many high intensity wind gusts. These can cause overloading of some elements due to load sharing to cope with the load shed by the damaged elements. Repeated gusts can therefore cause repeated failures.

The failure process is similar to that described in (i) above, though here the initial failure is not necessarily limited to fatigue. A single large wind gust may place local pressures on the roof that exceed the capacity of a single structural element, say for example a rafter anchorage. After failure of a single element, the load it carried is redistributed to other similar elements. Overloading during big gusts or subsequent gusts can cause other elements to fail. In our example, the adjacent anchorages may also fail as the battens bent to carry the load.

As more fasteners fail so the magnitude of the overload increases and other elements fail progressively more rapidly. It is quite common for the complete roof to be removed from the house as a single unit by this type of failure.

(iii) Effects Of Debris Damage

In Section 2.4, it was indicated that the internal pressure within a building was greatly influenced by the position of dominant openings. The sudden appearance of a dominant opening on a windward wall may double the net uplift on the roof.

Discussions with occupiers of buildings have frequently yielded case histories that have these characteristics. Houses can be struck by pieces of airborne debris such as a branch of a tree or a piece of roofing torn from another house. In these cases cladding damage may result. Where doors or windows are hit by big pieces of debris, catches or hinges can fail or glass can be broken. As the windward wall of the house is the most susceptible side to damage by debris, the hole caused by the debris admits positive pressure to the house.

The positive internal pressure can cause sudden overload of many roof structure elements leading to the apparently simultaneous failure of a very large number of fasteners. In these cases it may be nearly impossible to locate the first one to fail.

(iv) Damage Due To Lateral Loads

In contrast with the uplift failures described in parts (i), (ii) and (iii) above, failures due to lateral loads on houses can occur quite slowly. Occupants may be able to describe the failure in quite a lot of detail.

The more progressive nature of the failures by bracing components is due to two main effects:

Lateral loads do not fluctuate as rapidly as the uplift loads and hence build up more slowly. Ductile building components have a chance to respond to the slowly increasing load by extending and deforming prior to their failure.

Also the mass of the structure and contents that is supported by most lateral load caying elements is much greater than that of roof structure elements. This builds inertia into the system and prevents the house from responding rapidly to failure of individual components.

Frequently, however, the net effect of lateral failures is very severe damage as nearly all parts of the building are damaged by a less than graceful trip to the ground, as shown in Figure 30.

(building has been moved sideways off footings)

Figure 30 Damage Due To Lateral Loads

3.4 Characteristics Of First Failure

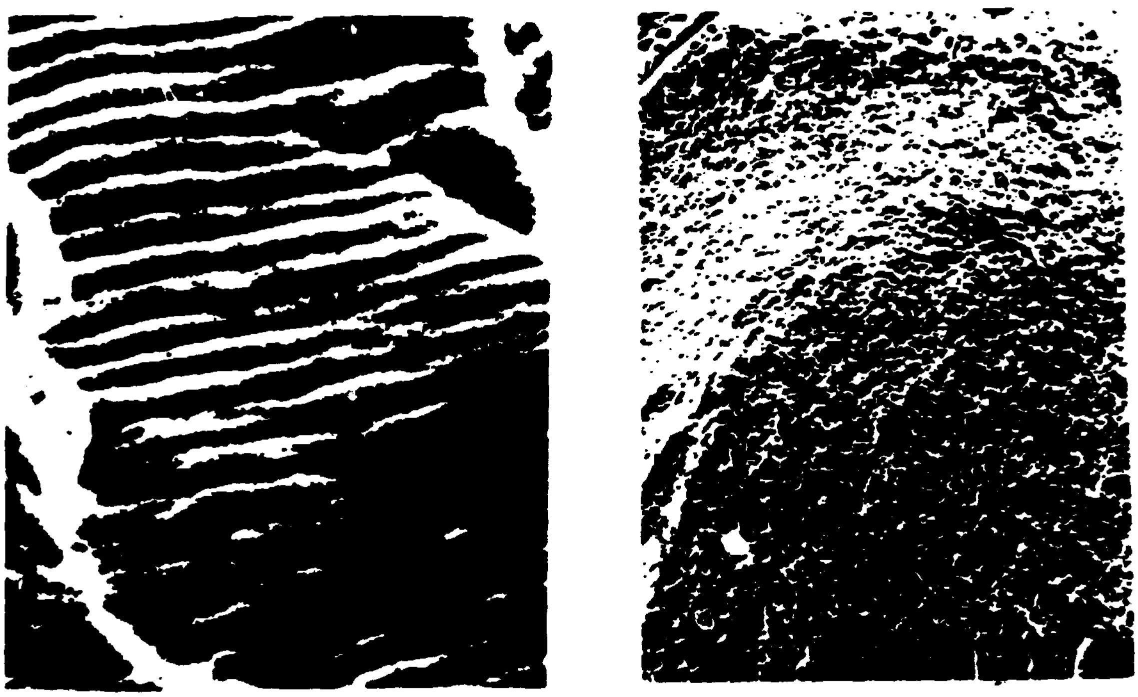
As indicated in Section 3.3, first failures in structural components of housing have some subtle characteristics. These can be used to confirm a suspected failure mechanism postulated for a given structure.

3.4.1 Fatigue In Metal Elements

Metals that have been damaged under repeated loading often fail due to propagation of fatigue induced cracks. The surface of these cracks show "striations", a series of small hills and valleys across the line of the crack. The striations are caused by the varying stress conditions within the metal during the propagation of the crack.

They contrast with the generally smooth surfaces of metal elements broken with application of a single large load. When the failure surfaces are fresh, a fatigue crack surface is dark grey and mottled in appearance, whereas an overload failure surface is bright and shiny. This differentiation can rarely be relied upon after typhoons because wind and water serve to discolour any failure surface very quickly. However, most fatigue cracks produced by typhoons have striations about half the size of fingerprints and can easily be seen using a 10x hand lens or magnifying glass.

A magnified impression of the two types of failure surface are shown in Figure 31.



(a) fatigue failure

(b) overload failure

Figure 31 Failure Surfaces In Metals Due To Fatigue And Overloading

By examining tears in metal elements such as claddings or flashings it may be possible to identify one or more that show striations which indicate a progressive fatigue failure at that point early in the damage history.

Conversely the examination may show these tears produced by overload activity and hence not associated with early failures.

3.4.2 Nail Withdrawal

Timber connections that sustain high loads may fail due to nail withdrawal. Those joints that fail early in the history of the damage can generally be identified by examination of the nails themselves.

(i) Nails Ordinarily Loaded In Direct Tension

Some nails carry wind loads in direct tension. Roofing nails are a common example. Generally the first nails to withdraw from the timber are pulled straight out and the nails remain straight. Where the nails pass through thin steel there is little tearing in the metal at the nail head with these failures.

Once the roof sheeting or other structural components are partially freed by the removal of a few nails, then the extra movement allowed in the released component can cause bending or twisting. This almost invariably causes bending of the nails.

Generally straight nails and clean holes are associated with early failures. However, elongation of the holes at lap joints in metal roof sheeting can indicate transmission of lateral loads rather than the withdrawal of the nail subsequent to the first failures. Nails at lap joints of sheeting should not be used in a definite analysis of nail withdrawals.

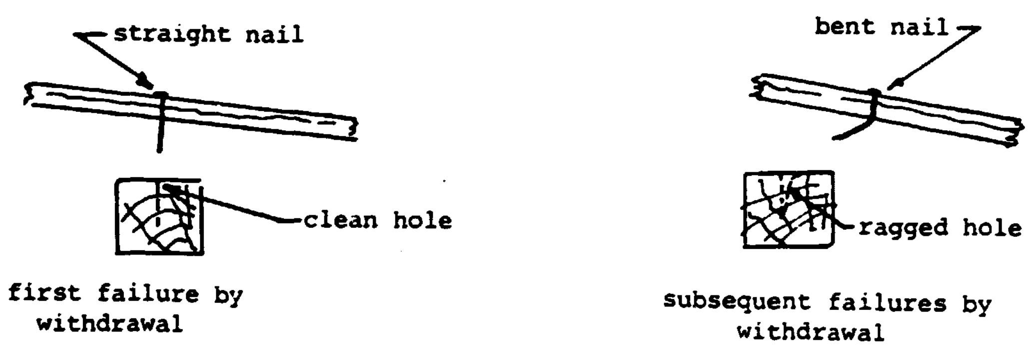


Figure 32 Withdrawal Of Nails

(ii) Nails Ordinarily Loaded In Shear

This type of joint includes shear nailed connections and many others used in light timber framing as well as those used to secure structural claddings to wall frames. Two modes of failure are commonly observed and both incorporate bending of the nail.

In one, the nail bends and then withdraws from the timber into which it is driven. In the other, the nail bends and then the head tears through the material fastened to the timber. The mechanism of failure observed in any particular case is a function of the relative strengths of the materials joined by the nails rather than the time at which the failure occurred. For shear nailed connections a third type of failure has been observed in timber prone to splitting. This is caused by splits often established in driving the nails, but which open suficiently to allow the timber to part at the split under wind load. Figure 33 illustrates the three failures.

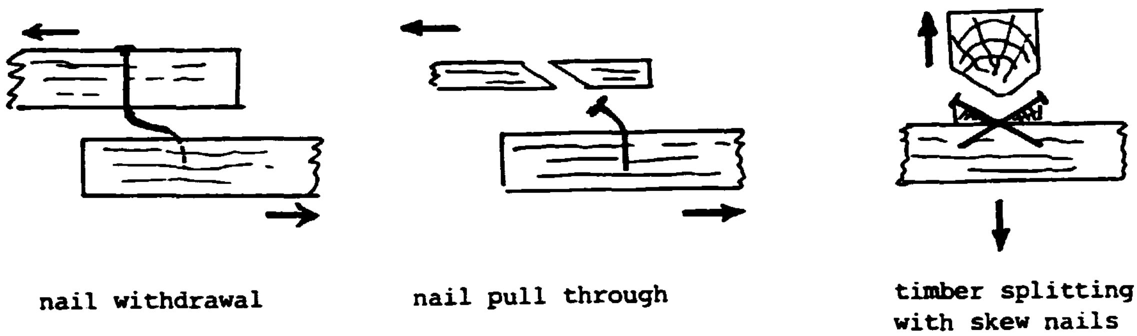


Figure 33 Failure Of Nailed Joints In Shear

It is very difficult to differentiate between the failures that ocurred early in the history of the damage from those that occurred as the failure was spreading due to overload.

Some clues may be derived by the way in which the nail is bent, although this is not as definitive as the identification for metal elements or nails in withdrawal. Early in the damage history the joined pieces are not usually separated so that the nails must bend about a very tight radius. This in itself forces the joined pieces apart. Subsequent nails have partially withdrawn already and therefore are bent over larger radii.

3.4.3 Timber Failure

It is unusual for timber failure to be the initiating damage in a chain of failures as discussed for nails and metal elements in the previous subsections. In many cases, prior failures of connections cause excessive bending moments in timber framing which lead to later failures.

In most cases, the timber in houses is dried and as a result has a lower tensile strength than compression strength. The flexural failure of timber is generally initiated by tensile failure and as a result is brittle and produces a jagged separation. This is more common where the timber incorporates defects such as angled grain or knots in the vicinity of maximum moments.

The order of timber failures can be identified by retrospective analysis. In many cases, however, due account must be taken of the presence of defects. They have two effects:

> They lower the stength of the timber although this effect can be estimated from either documented failure properties of the timber or by factoring the allowable strength of engineering grades of timber rather than the select grades of timber.
>
> They also can cause the failure position to be located at the defect rather than the maximum moment position.

Note that in order to be able to use retrospective analysis of timber it must be possible to identify the species of timber in which the failure occurred. If the occupier built the house they may be able to assist in the identification of the timber, otherwise if it is a crucial analysis samples can be taken for checking against a key. Most timber research facilities support identification keys.

3.4.4 Failures In Bamboo Or Mudbrick

At this stage, the author does not feel competent to comment on the failures in these materials. This presents a challenge to future damage assessments.

3.5 As essment Of Damage To Housing

In the assessment of damage to housing, some calculations will be required to determine wind loads on the house as a whole and on structural components. However, there is no necessity to perform these at the time the inspection is made. Provided an adequate record of the damage is taken by notes or tape recording and photographs, the calculations can be performed at a later more convenient time.

Care should be taken that either the first failure is identified or sufficient detail is taken to enable its determination away from the site. It is generally not possible to return to reassess damage.

Further detail on assessment techniques is provided in Chapter 5.

4.0 STRUCTURAL ACTION OF LARGER BUILDINGS

The structural action of larger buildings can be much easier to ascertain accurately from damage assessments when compared with that of small houses. Structural engineers are frequently occupied with the design or analysis of larger buildings and often feel more 'comfortable' with them. However, it is much more difficult making generalisations about their behaviour.

Few large buildings incorporate structural cladding. Most have a large skeletal structural frame which receives load from the cladding and carries it to ground by frame action or through specially constructed bracing members. Some recent buildings clad with specially designed metal diaphragms rely on a stressed skin membrane action to carry loads to ground. There are some reservations about the capability of such systems to resist typhoon loads (Boughton, 1981).

4.1 Buildings Without Structural Cladding

In large buildings without structural cladding it is generally possible to identify the structural members and their intended function even in severely damaged structures.

(1) Large Members

Principal structural components may include portal frames, braced frames or unbraced multi-storey moment carrying frames. The four most commonly used building materais for these structures are steel, concrete, timber and masonry. Often all four are incorporated in various components in a single structure.

The principal structural components are often large in size and span large distances. Their analysis can be accomplished using moment distribution techniques, structural analysis computer programs or in statically determinate cases, by direct application of the principles of equilibrium.

The larger members frequently support either smaller members or floor slabs. In multi-storey structures, the primary design constraint is frequently gravity forces due to floor loads. However, large lateral forces associated with typhoon winds can cause large lateral deformations which can influence the structural action of larger buildings.

Where damage to large members has been caused by typhoon winds, it can render the building completely ineffective for its intended purpose. This is not a common occurrence due to a number of aerodynamic effects. Gusts are generally of finite size and cannot completely engulf large buildings. This means that the total load on a large building and the parts of large buildings supported by the primary structural elements is frequently overestimated in codes. The members themselves are often overdesigned. Many damage reports have shown examples of large portal frame industrial buildings in which the frames remain, but the purlins and cladding have sustained very severe damage.

In contrast to the overdesign of the frames, cladding and cladding support members are often underdesigned. This is due to the turbulence induced high local suctions near edges of the building and the ridge. These very isolated effects only affect the cladding and purlins but do not cause any increase in load of the principal members due to the small size of the local pressures compared with the area supported by the larger members. The underdesign of cladding means that it has a much higher probability of failure than the portals. Failure of cladding causes a change in the aerodynamics of the structure which reduces the load on the larger members. Figure 34 shows the most commonly observed type of damage in large industrial buildings in which the major structural components have sustained minimal damage.

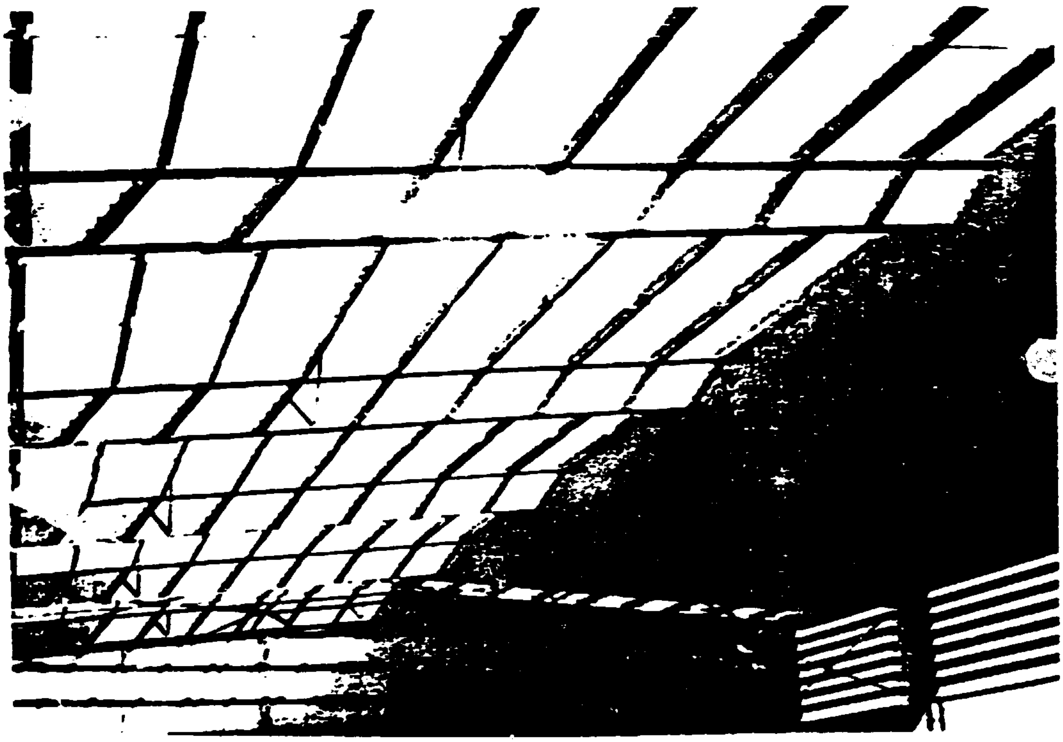


Figure 34 Damage To Large Buildings

(2) Bracing Members

Bracing memb..s in large buildings are generally designed to lend rigidity to the structure during construction and to carry lateral wind loads in the completed structure. Frequently bracing members are slender and are designed as tension members. Generally crossed bracing members are used and the compression member is expected to buckle out-of-plane and be ineffective in resisting load.

The loads carried by bracing members can be determined using the techniques presented in Section 3.1.2. Wind forces over the area that contributes load to the bracing can be calculated and then resolved in the direction of the brace using its angle to the horizontal.

Damage can result when connections do not have sufficient stiffness and allow excessive deflection. The deflections can damage partitions or panels. More severe damage can result if the connections fail. Partial or total collapse of the structure can follow.

Failures due to local and global buckling of bracing elements have also been observed where cross bracing has not been used and the bracing members carry compression.

(3) Purlins, Girts And Other Cladding Supports

These members frequently span larger distances in large buildings andso are bigger members themselves. The longer spans also mean that bending moments in the purlins and the forces on the conneciltion at each end are larger. The calculation of loads on purlins follows the same procedure as presented in Section 3.1.2. Girts and other wall cladding supports function in a very similar fashion to purlins and the methods outlined for battens in Section 3.1.2 are equally applicable to them.

A complication can arise in large buildings that use cold formed steel purlins and girts. Where the cladding has suction forces on the external surfaces, the compression flange of the purlin is unsupported on the inside of the building. Lateral buckling can occur under these circumstances. Cases have been observed in Australia where the cladding has remained fastened to the purlins but they have buckled. The building profile is therefore distorted, but it still remains functional. Roofing and purlins must both be replaced following this type of damage. Effective design of replacement items should include bridging pieces that will brace the critical internal flange.

(4) Cladding

The problems that commonly arise with cladding are identical to those covered in Section 3.

Frequently large buildings protect very valuable equipment and or records. Leakage of cladding can cause significant damage to the contents of the building. Discussions with the owners or occupiers of large buildings will reveal this type of problem. It can present a very significant financial burden to the community if stock held in warehouses, foodstuffs, agricultural products, or financial records are damaged by water ingress during a typhoon. While the damage is not as spectacular as loss of roofing or major structural damage, the cost of water damage can be very high and so it warrants inclusion in a damage assessment report.

4.2 Buildings With Structural Cladding

The inspection, analysis and design of these buildings is a highly specialised branch of structural engineering. Stressed skin designed buildings can be recognised from the inside by the fact that no bracing members are used in the structural framework of the building. They may also be recogised from the outside by the presence of large numbers of fasteners along lap joints between adjacent sheets, particularly near the end of the building.

Seam fasteners, illustrated in Figure 35, are an essential feature of stressed skin design buildings as they transmit shear forces within the roof diaphragm across the lap joints.

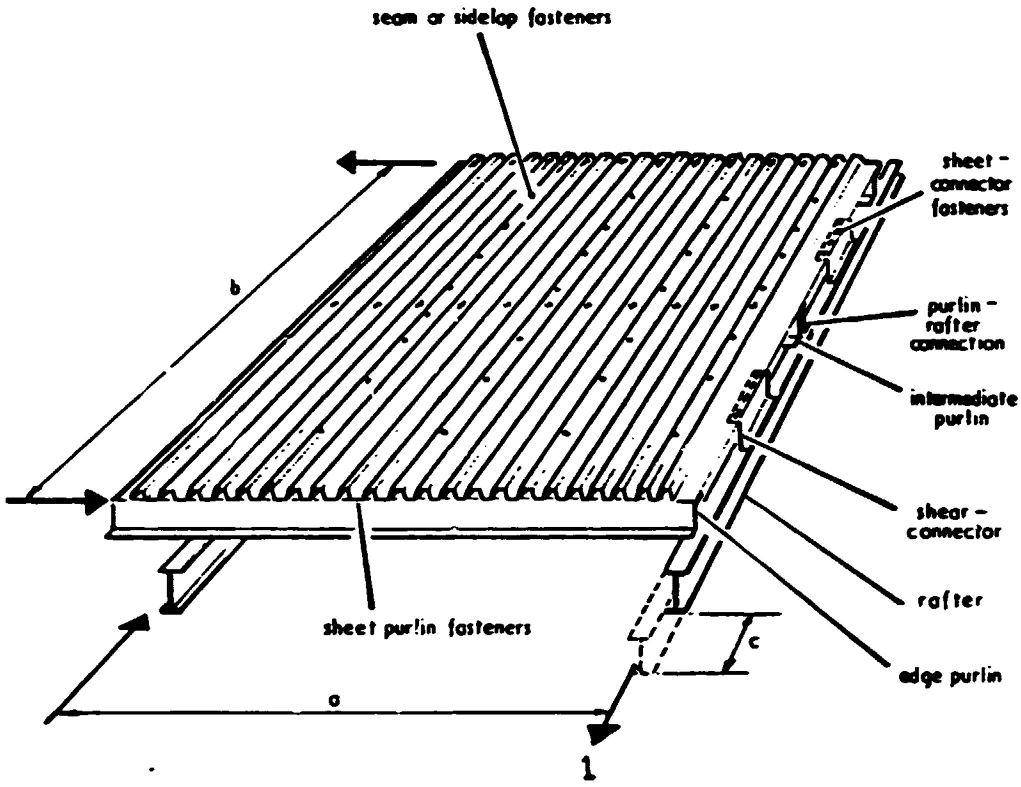


Figure 35 A Typical Stressed Skin Panel

A detailed treatment of stressed skin diaphragm buildings has been published by Davies and Bryan (1985). This book, or a similar publication should be used as a guide to the assessment of stressed skin diaphragm buildings.

4.3 Observation Of Damage

The significance of even minor damage to large buildings has already been mentioned. Some large buildings such as schools and other government offices have special post disaster functions in that they provide shelter for the homeless or may be required as administrative centres for relief operations and reconstruction. Buildings that fulfil these roles, as well as hospitals, warrant assessment even if no obvious damage is visible. Their function is important enough to deserve an inspection to locate damage which while not obvious, may cause a reduction in structural performance if another typhoon should cause high winds to load the building again later in its lifetime. These types of inspection to buildings with little or no damage are known as "residual life assessments".

Results of residual life assessments should be sent to the operators of the building checked as soon as possible, to enable remedial work to be commenced.

4.3.1 Number Of Observations And Accuracy Of Assessment

Unlike housing where similar designs tend to be repeated often within an area, larger buildings tend to be designed and constructed as required. Any town may contain a few large buildings but they may be of quite different design from each other. Those that have been damaged should each be assessed independently.

The same inaccuracies in wind speed and wind load estimation that were mentioned in connection with housing, apply to larger buildings, however the smaller number of buildings generally available for inspection means that there will be less confidence in the conclusions for larger buildings. Where the main type of damage observed was cladding, it may be possible to group the findings with observations of similar damage in housing and increase the confidence level of the conclusions drawn.

4.3.2 Identification Of First Failure

The principal structural elements such as frames draw wind load from a large surface area of building while purlins and sheeting fasteners attract load over small surface areas. This enables the two types of members to be considered as aerodynamically independent. and is a major difference from the problems associated with housing.

A number of observations have been made of buildings in which the cladding and purlin systems have sustained severe damage, but the principal members have received only damage to paint. In these cases it is easy to associate the first failure with the cladding and purlin systems. A detailed examination of the roofing debris will then enable the first failure to be attached to either the roofing or the purlins.

There have been fewer observations of buildings in which the cladding has remained intact but the principal structural members have failed. In these cases the first failure can be associated with collapse of the main structural system of the building. Such a failure is the result of formation of a plastic mechanism, in which a number of local failures must form. Possible collapse mechanism are illustrated for simple portals in Figure 36. The location of plastic hinges, or areas in which the main concrete or steel members have yielded or crumpled has been shown.

The analysis of the behaviour of these mechanisms can be accomplished using standard plastic analysis techniques. Later access to the engineering drawings of the structure will assist in the analysis. This is particularly so with concrete structures as frequently the number and location of reinforcement bars cannot be obtained from site inspection.

The independence of the structural behaviour of main structural frame members from the behaviour of cladding makes identification of first failure easier. Any of the techniques outlined in Section 3.4 for finding first failure in cladding and fastening systems could be applied equally well to the assessment of larger buildings.

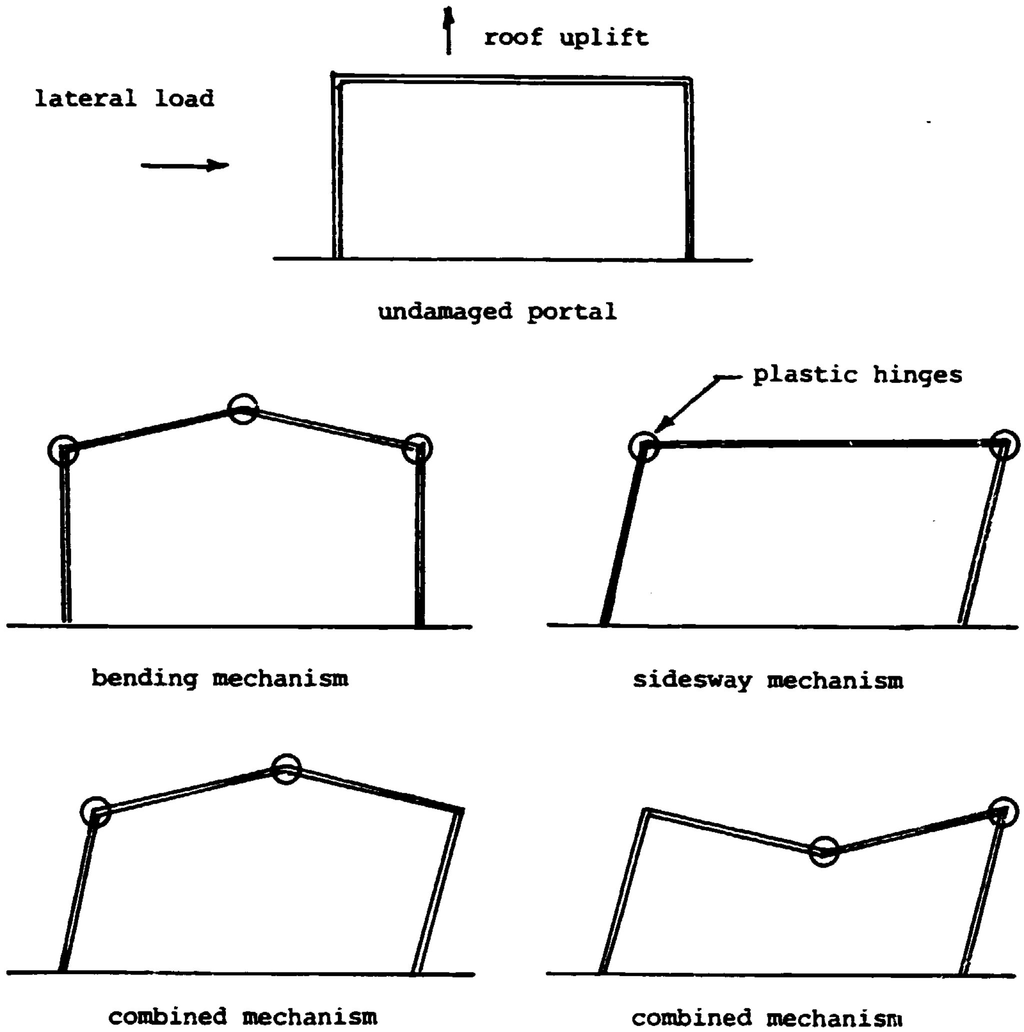


Figure 36 Possible Plastic Failure Mechanisms For A Simple Portal Frame

4.4. Assessment Of Damage To Large Buildings

In the assessment of large buildings there may be problems that can only be solved by examining engineering drawings. This particularly applies to lined buildings such as schools and hospitals in which structural details can be hidden. Under these circumstances the analysis process following the initial inspection could be quite time consuming and can only be accomplished away from the site. Extensive and good photographs of not only the whole building, but connections and members within the damaged structure should be taken.

The presentation of the assessment is covered in Chapter 5.

5. ASSESSMENT TECHNIQUES

This chapter addresses the issues of grouping the data, evaluation of the individual building assessments, presentation of the report and production of recommendations for future construction practice.

5.1 Scale Of Events To Be Assessed

A major influence on the use to which the results of the assessment will be put is the scale of the event.

Often very useful information can be obrained from small scale typhoons in which not much very severe damage has been caused. Where most of the damage is simple structural damage, its assessment is easy and the first failure is almost indisputable. The inspection of damage is usually easy and quick and the later analysis of the data can also be accomplished quickly. Reports from mild typhoons can therefore be produced within a couple of months which means they are published while memories of the event are still fresh. Some very useful conclusions can be drawn as a result of the assessment (Boughton, 1987a).

More severe typhoons generally produce more widespread and severe damage. The practical problems associated with movement through an area recently affected by a severe typhoon makes gathering the information more difficult. Often the severity of the damage makes the collection of data a long process and the analysis of each severely damaged building can also be a time consuming task. Much sorting of the data will have to be performed and the report will necessarily be large. The time for its production may be many months. It will, of course, contain a much wider range of conclusions than the report covering mild typhoons.

Both events can be used to pinpoint weaknesses in buildings. Details that cause simple structural damage in mild typhoons will almost certainly cause very severe damage in severe typhoons unless they are systematically strengthened. There is therefore very good reason for implementing damage assessment for each and every typhoon that causes damage to buildings. Where the extent of the damage is not very severe, the cost of the assessment is not large, but the potential benefit the assessment offers is still substantial.

In the presentation of the data obtained in the course of the assessment, the work should be referenced to the intensity of the typhoon that caused the damage. This will enable the information to be usefully applied by people in other locations. The intensity of the typhoon can be measured in a number of ways. Perhaps the most universaly acceptable measure of intensity is the central pressure. For the purpose of evaluation of wind loads on buildings wind speeds must be obtained. A number methods have been presented in Chapter 2 of this work that will enable the determination of wind speeds from meterorological records or from observation of the effect of the wind.

WHERE WIND SPEEDS ARE PRESENTED, THE TIME SCALE OF THE SPEED MUST ALSO BE QUOTED

For example, a ten minute mean wind speed of 40m/s represents a much more severe loading than an event in which the peak three second gust was 40m/s, but it is less severe than one in which the mean hourly wind speed was 40m/s.

Likewise it is important to quote these wind speeds as equivalent speeds for the internationally recognised standard environment for wind speed measurement.

A TEN METRE HEIGHT IN FLAT OPEN COUNTRY
(This is similar to that encountered at large airports).

The correct interpretation of wind speed data is a fundamental building block upon which the remainder of any assessment rests.

5.2 Information Required For Each Building Assessed

The aim of an assessment of every building is to determine the cause of structural failure in the building and the load at which it occurred. These two data can then be used along with observations of the structural details used in that buildings and other similar buildings to comment on the performance of the building.

It is appreciated that the inconvenient living and working conditions during the assessment and the need to inspect many buildings very quickly will place practical limitations on the amount of time to be spent at any one site. To that end, information must be systematically recorded to allow later evaluation and analysis.

5.2.1 Buildings

The information is most conveniently taken in the following order:

(1) When approaching a building the extent of the damage should be noted and the building described by some convenient and obvious feature.

An example of this type of identification may be as follows:

"Small Plywood house with roof structure completely mssing - no sign of any of the roofing, some minor damage to walls and extensive water damage of contents. Red curtains hanging out of windows and belongings drying on the lawn".

This type of description can be made from the roadside outside the block and provides a rough description of the size of house, materials and extent of damage. This can be incorporated in a statistical treatment of the damage later. The reference to curtains and the belongings will enable the house to be identified in photographs.

(2) From this rough assessment it should be decided whether there will be much value in conducting a detailed assessment.

If many houses of that type have been checked already or if clean-up activities are already well advanced, it may be decided not to inspect it. In that case "no further inspection and no photographs" should be attached to the notes for that house. On a sheet of paper the address of the house should be recorded with a note saying "damaged but not inspected", an appropriate abbreviation would be DX.

If, on the other hand, the type of damage is of interest then a photograph of the house showing the damage and in which the distinguishing features of the description are clearly visible. This will enable matching of the notes to photographs.

EACH TIME A PHOTOGRAPH IS TAKEN A NOTE SHOULD BE TAKEN INDICATING THE ANGLE FROM WHICH THE PHOTOGRAPH WAS TAKEN OR FOR DETAILS, WHERE THE DETAIL WAS LOCATED IN THE HOUSE.

At this point too, on a sheet of paper, the address of the house should be recorded with a note indicating that it was "damaged and inspected". "D" and a check mark would be an appropriate abbreviation.

(3) After gaining permission to enter the property, the damage can be examined and described in greater detail. An appropriate order would be similar to this for the above example

- roof removed by disconnection of the entire roof structure from the top of walls when the wind was blowing from the North-West.

  (A photograph showing the terrain to the North-West should be taken and appropriate notes made about the proximity of other buildings, the vegetation and topographic features to the North-West).

- damaged roof found in next door neighbour's yard.

  (A photograph of the condition of the roof should be taken and again notes made).

- condition of roof is reasonable, some distortion due to impact with ground and a tree, but roofing to batten fasteners and batten to rafter fasteners largely intact.

  (Some more notes and perhaps a photograph of the fasteners in the roof structure that appear to have been effective may help the evaluation).

- roof anchorage details - two skew nails per rafter failed in most cases by splitting of rafter timber. Only one nail used in the rafter anchorage at North-West (windward) corner.

  (A photograph of the details - in particular the one at the North-West corner with only one nail hole should be taken).

- discussions with occupants revealed that all of the windows were barred and none were damaged.

  (Notes should be taken with respect to the leakiness of the house).

- Dimensions of the house should be estimated.

  wall height
  length of house
  width of house
  height of floor above ground
  slope of roof
  eaves overhang
  age of house
  identity of the builder

  In our case the important detail is the roof anchorage detail, so the following notes may also be made.

  species of rafter timber
  size of rafter
  species of top wall plate timber
  size of top wall plate
  size of nails

- Finally, any other interesting or unusual features may be noted. (These may include - a particularly effective window barring system with details and photographs).

- If any information is to be passed on to the emergency services it should be noted on the paper that has the house address. For example "No access to drinking water yet".

The information and photographs listed above will enable a reasonably detailed analysis and assessment of the performance of the structure. It is to be noted that only sufficient dimensions were recorded to find the wind loads on the whole house and on the particular element that was identified as the one that caused the failure. This is quite sufficient for this case in which the damage would have been classified as simple structural damage. More detail would have been required for local damage or very severe damage.

(4) After leaving the property an interpretation of the damage can be made. It is most important that this be done out of earshort of the owner/occupier who may be offended by your comments if he was the builder. Alternatively an argument between the owner and the builder may be fuelled by your comments.

To continue with our example, an appropriate comment may be:

- due to the use of one nail at rafter anchorage in the windward corner anchorage failed.

- subsequent overloading of other anchorages caused their failures.

- need to check capacity of anchorage with one skew nail and capacity with two.

This then will complete the data gathering process for that house.

5.2.2 Simple Structures For Wind Speed Analysis

In the examination of simple structures for the determination of wind speed, the following points should be noted.

It is just as important to analyse undamaged structures as it is to analyse damaged ones. The small amount of time taken to measure up good ones may well pay dividends, as they will enable the determination of upper bounds to the peak gust speeds at that locations.

Please note that while it may be possible to estimate dimensions on buildings the importance of these simple structures means that they must be measured.

(1) Close examination for scratches, timber residue that may indicate that debris struck the structure. If clear of debris damage then the investigation can proceed.

(2) Description of the simple structure again with identifying information eg "speed limit sign next to a school building".

The damage should also be described.

(3) The orientation of the face of the structure should be measured with a compass. Be wary about standing under power lines or too close to metal signs when using a compass. The orientation of the wind damage in the vicinity of the structure should also be noted.

(4) Using the orientation of the wind damage near the structure as a guide to the direction of maximum winds, if the wind was approximately perpendicular to the face of the structure the investigation can proceed.

(5) Photograph and note terrain and topographic features upwind of the structure.

(6) Measure the simple structure with a tape so that the calculations outlined in Chapter 2 can be performed.

5.2.3 Daily Checks Of Damage Surveyed

The daily checks indicated in Chapter 1 of this work will primarily use the information on the paper note summary ie address and whether inspected. The information for emergency services can be collated from these notes and passed on to the appropriate authority.

Good communication between the inspectors will ensure that houses will not be assessed twice, and that a balanced view of the damage is being obtained. Discussions can also assist in the decisions as to which structures to check and how much information should be recorded. It will also allow a check on the number of simple structures assessed. Where possible, at least four should be used to determine wind speeds in any locality or town.

5.3 Processing Information For Each Building Assessed

The steps in processing information for each building assessed can be subdivied into practical and technical steps. Generally the practical steps precede the technical steps.

5.3.1 Organisation Of Data

Where notes in the field were taken using a tape recorder the tapes should be numbered sequentially. They should then be roughly transposed so that a paper working copy is obtained, the tapes can be kept as a reference, but most of the analysis work will be done from the transposed notes.

(1) Assign each structure checked a number. This number should indicate the following information.

Inspector
Building Number
Typhoon

eg B14C6 indicates Boughton as Inspector
14th building he inspected
Cyclone "Connie" in 1986

Using the distinguishing features of the building, the photographs should be matched with the notes. Every photo should have the building number marked on it and the notes should also be marked with that number.

(2) The type of failure and other interesting features should be highlighted in the notes. Highlighting pens or coloured pencils are excellent for this. This will assist in later collating of information.

(3) From information on topography, terrain and wind speed, the wind load on the building should be determined. This, of course, assumes that all the simple structures have been analysed first to determine the peak gust wind speed using techniques outlined in Chapter 2.

(4) Determine the maximum wind load on the element noted as the first failure using techniques described in Chapter 3, and include that figure in the notes.

(5) If design information is available the design strength of the broken element can also be noted.

(6) Steps (4) and (5) can be repeated or any other elements noted as being interesting or significant in the structural behaviour of the building steps .

5.4 Grouping Of Information

While the data collected and processed as indicated above represents a valuable resource, it can have its full potential realised by grouping and reporting it in an appropriate way.

5.4.1 Grouping By Construction Type

Very rarely are damage reports completely exhaustive so assessments should be wary about reporting on the relative merits of construction types. This type of grouping can only be used to conclude that one type is more successful than another if all houses are reported damaged or undamaged and if sample sizes are similar. Very small sample sizes cannot give results to which much statistical confidence can be attached.

Grouping by construction type can show a number of important trends in the data.

(i) Failure mode may be well correlated with construction type.

In some cases particular types of construction may all show the same or similar types of failure. This has therefore identified a systematic weakness in that type of construction.

(ii) Damage severity may be well correlated with structure wind speed within a given construction type.

Over a wide range of terrain and topographic conditions, a variety of wind speeds at the structure height will be found. In many instances there will be a good correlation between damage and wind speed. This can be used to establish the wind speed at which that type of construction ceased to behave satisfactorily.

5.4.2 Grouping By Type Of Builder

This will enable the identification of any problems in workmanship. A suggested categorisation may be owner/builder, self employed builder, small construction company (less than ten employees), large construction company (more than ten employees).

The results of this type of grouping can be quite surprising and will assist in the formulation of housing policy.

In areas where building inspection during construction is performed, that could be included as an additional category i.e. inspected construction.

5.4.3 Grouping By Failure

This is possibly the most useful type of grouping. It has the same effect as increasing the sample size for structural tests in the laboratory.

The information produced by this grouping is made complete if some buildings that sustained minimal damage have also been assessed. That enables the evaluation of an upper bound to the failure load and complements the lower bound obtained through the analysis of damaged structures.

By comparing the failure loads for elements of the same type and configuration, an estimate of their capacity for wind loadings can be made. Where design load information is available, the failure loads can be compared with design loads. If the failure load is well in excess of the design, load it indicates that the element has been used inappropriately. If the failure load is less than the design load, it indicates that the currently quoted design load may be in error and further work must be performed on those details.

Where no design loads are available, the failure loads can be used to estimate appropriate design loads using suitable factors of safety.

Design load = k (95% lower confidence limit of failure load).

k = a factor usually between 0.5 and 0.66

In performing this grouping it must be noted that materials and geometry must all be similar. For example, a nailed batten joint using two skew nails should not be grouped with a similar joint using only one skew nail.

To enable this type of grouping, details on the failed elements must be accurately taken. In some cases nail size is critical, particularly where withdrawal has occurred.

5.5 Identification Of Problem Areas

The grouped data may make some trends obvious. These trends can be traced to specific problem areas that recur in a significant way during the damage investigations. They can be broadly categorised as workmanship problems, design faults or material deficiences.

5.5.1 Workmanship Problems

Where failure loads are consistently less than accepted design loads or where failure has occurred because of incorrect installation of usually reliable details, then poor workmanship may be the cause of the damage.

Some common examples of this problem is omission of nails in joints, nails that miss the timber they are meant to be driven into, omission of fasteners in roofing, use of large purlin or batten spacings, lack of reinforcing in block or brickwork.

5.5.2 Design Faults

In some cases details, consistently used in a locality may not be appropriate for the loads expected in typhoons.

This may be due to poor design or even lack of design and the widespread use of these details has made them acceptable as good building practice In some cases poor design details may have been incorporated into building regulations.

The results of damage assessments present very good evidence in support of the case for changing regulations or designs.

5.5.3 Material Deficiencies

In some cases, observations may have showed that failures were associated with rotten timber or timber that had been subjected to insect or termite attack. Rusted steel or corroded steel connections are also a common cause of failures. In some cases, poorly compacted concrete or weak mortar may be associated with the failures.

In these cases the deterioration may be related to the age of the structure or to the environment. In either case the information is valuable in establishing the expected life of particular details in particular areas.

5.6 Preparation Of Assessment Report

After the processing of data, the grouping and assessment of the data, the report can be prepared. The grouping of the data will have enabled a number of conclusions to be drawn on the performance of buildings and components of the buildings.

The report should be written in a way that leads directly to those conclusions. Raw data should not be included in the report except to support points made in the text of the report. Some raw data can be used to draw up tables, and observations should be illustrated by photographs or sketches wherever possible.

Suggested headings are as follows:

(1) Introduction - This should describe the locality in which the assessment was made and enable it to be identified with respect to the country in which it is located. It may include geographical comments on the type of buildings and communities in the affected area and may allude to a previous history of typhoons.

(2) Meteorological Information - This should include meteorological records where they are available and the results of wind speed determination from any means used. It should give an indication of the maximum estimated wind speed at all locations in the damaged area and an estimate of the direction of the maximum gusts.

(3) Assessment Of Building Performance - These should be grouped in a way that will maximise the impact of the conclusions.

For example - buildings with damaged roofs
- damage to timber structures
- damage to large industrial buildings

(4) Implications Of The Building Performance - This section relates accepted building practice to performance, and may indicate areas in which specific problems need to be addressed.

(5) Recommendations And Conclusions - These will be concisely stated practical activities that can be performed to improve the performance of buildings in future typhoons.

6. CONCLUSIONS

This work has set out a procedure to be followed to produce a comprehensive assessment of building performance in typhoons based on inspection of damage caused by the passage of a typhoon.

1. Assessments of buildings can improve the understanding of their structural performance and quantify failure loads without recourse to expensive and timeconsuming laboratory tests.

2. Comprehensive assessments of the type outlined in this work require thorough preparation, planning and organisation. It is essential to perform the inspection quickly and systematically as soon as possible after the event of a typhoon.

3. The damage must be related to meteorological information to give it a quantitative reference.

4. In the performance of an inspection of a building it is very important to differentiate between the first elements that failed and those that failed subsequently due to overloading.

5. By calculating wind loads on the first element to fail, the assessment can be related to design loads and test results.

6. Failure loads and performance information produced by systematic assessment can be used to improve building designs or upgrade building regulations.

7. It is recommended that member nations immediately draft plans for implementation of assessment programmes following typhoons, designate area supervisors and assessment team members, and establish liaison with emergency service personnel. Funds should also be set aside for assessment activities.

7. ACKNOWLEDGEMENTS

The author wishes to acknowledge the inspiration and training in this area offered by my colleagues and former workmates, Associate Professor George Walker and Mr Greg Reardon of James Cook University in North Queensland.

The author is also grateful to Curtin University of Technology for assisting in the publication in particular to Mrs Jacquie Kelly and Mrs Marj Withnell, who worked hard and fast to produce the typewritten final copy.

The role of the Regional Secretariate to the UNDP/UNIDO Regional Network in Asia is also acknowledged. The organisation of the Seminar/Workshop provided the impetus to the publication of this work.

8. REFERENCES

Amadore, L.A., Bucey, J.F., Talib, B.D. and Yanga, S.O. (1985) "Preliminary Typhoon Damage Scale In The Philippines", proc. Joint AMS/PMS International Conference In Applied Meteorology And Climatology, 18-22 March, Manila.

Atkinson, G.D. and Holliday, C.R. (1975). "Tropical Cyclone Minimum Sea Level Pressure - Maximum Sustained Wind Relationships For The Western North Pacific" Bulletin Of American Meteorological Society Vol. 56, No. 2.

Boughton, G.N. (1981) "Bracing Strength Of Corrugated Steel Roofing" Technical Report No. 8, Cyclone Testing Station, James Cook University of North Queensland.

Boughton, G.N. (1987a) "Investigation Of Damage To Structures Following Cyclone Connie" Civil Report 87/1, Curtin University of Technology, Perth, W.A.

Boughton, G.N. (1987b) "An Investigation Of The Behaviour Of Full Scale Housing Under Simulated Wind Loads", PhD Thesis, James Cook University of North Queensland (in preparation).

Bowen, A.J. (1979) "Some Effects Of Escarpments On The Atmosphere Boundary Layer", PhD Thesis, University of Canterbury, Christchurch, N.Z.

Cook, N.J. (1985) The Designers' Guide To Wind Loading Of Building Structures, Part 1, Building Research Station, Garston, England.

Davies, J.M. and Bryan, E.R. (1982) Manual Of Stressed Skin Diaphragm Design, Granada, London.

Georgiou, P.N. Davenport, A.G. and Vickery, B.J. (1983) "Design Wind Speeds In Regions Dominated By Tropical Cyclones", Journal Of Wind Engineering And Industrial Aerodynamics, Vol. 13, pp 139-152.

Gomes, L. and Vickery, B.J. (1976) "Tropical Cyclone Gust Speeds Along The North Australian Coast", Civil Engineering Transactions, Institution Of Engineers, Australia, Vol. CE18, No.2, pp 40-48.

Reardon, G.F., Walker, G.R. and Jancauskas, E.D. (1986) "Effects Of Cyclone Winifred On Buildings", Technical Report No. 27, Cyclone Testing Station, James Cook University of North Queensland.

Roy, R.J. (1983) "Wind Tunnel Measurements Of Total Loads On Mobile House", Journal Of Wind Engineering And Industrial Aerodynamics, Vol. 13, pp 372-338.

Standards Association Of Australia (1983), SAA Loading Code Part 2 Wind Forces, AS1170-II(1983), Sydney, Australia.

Tryggvason, B.V. (1979) "Computer Simulation Of Tropical Cyclone Wind Effect For Australia", Wind Engineering Report No. 2/79, Department Of Civil And Systems Engineering, James Cook University of North Queensland.